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Water and Government

By Paul V. McNutt

OUT into the parched countryside of Mississippi in the fall of 1933 went agents of T. V. A.'s power division. T. V. A. was getting ready for the extension of rural lines which would bring the kilowatts of Wilson Dam to the farmer's door. T. V. A. wanted to find out about the demand—whether the people along these rural roads wanted electric power, and, if they did, what they wanted it for.

There were many answers, for electricity on the modern farm has a myriad of uses. But next to light itself, there was one answer that came first to the tongues of 9 out of 10 farmers—"Water. Water for the barn. Water for the milkshed. Water for the cooling house. Running water." And to every farmer's wife there was always that same answer: "Water. Running water, piped through the house. Water that can be depended upon all the year around. Water which will flow at the touch of a faucet."

You will find that same cry for water written large in the annals of the Rural Electrification Administration. Here, indeed, is a program that makes every farmer's wife a saleswoman and finds its arguments emerge instinctively from her passion for cleanliness.

Many are the influences, malign or maligned, that have been charged with responsibility for the exodus from farm to city—the

A paper presented on April 23, 1940, at the Kansas City Convention by the Hon. Paul V. McNutt, Federal Security Administrator.

movies, industrial jobs, bright lights, bad markets—but don't overlook the importance of that clear, colorless, tasteless fluid, which it is your purpose to purvey to his majesty the American public. If electricity goes eventually, like the postman, to the most isolated American farm, it will be, not the demand for ensilage cutters, but for water, that will take it there.

The story of the haughty little city girl who could not be fooled by the tall story that milk comes from cows, makes even city dwellers laugh; but nearly 70 million of these same city dwellers take their water supply for granted. When it doesn't taste right, they complain. Should it carry disease in a dramatic and epidemic fashion, they may be roused to action, or even appropriation. Should the supply itself be threatened, as it has been in some cities of the far Southwest, they may move political mountains to secure it. But for the most part they forget it. It's something that can be counted on, like the air, or the morning bottle of milk. There it is in the faucet, ready at the touch of the housewife's hand.

The creation of a municipal water supply is an engineering enterprise that is rich in dramatic accomplishment. It is a story of great dams, of deep wells, of tunneled mountains, of waters piped or canalized sometimes hundreds of miles across dry lands, of complicated treatment, plants and mains that honeycomb beneath cities.

Water supply is an old story, old as civilization itself. Every great civilization has made some approach toward solving it—Babylon, Greece, China. Rome was so well aware of the problem of pure water that it had two water supplies—one for drinking, one for bathing. The aqueducts, like the roads, remind us that Rome was not wholly devoted to recreation, politics, and war, that it was also a city of engineering genius.

One might say that civilization depends upon a safe and adequate water supply. As if to underscore that idea, the Middle Ages forgot the need for a safe water supply, and for 1,500 years civilization languished and epidemics swept Europe.

The construction phase of this problem is not a function of the Federal Security Agency. That phase is a concern of the Federal Works Agency and its predecessors. Some of the best W. P. A. projects employing relief labor have concerned water supply, drainage, and stream control.

Our interest in the Federal Security Agency is on the human side

of the problem. Our concern is in the health of the people who use water and in service to you who administer these enterprises.

Water Supply A Necessity

Civilization, I have suggested, is dependent upon an adequate and safe water supply. But civilization also is a constant threat to the dependability and safety of water supply. We are concerned with the latter aspect of that paradox.

Consider the uses of water in modern civilization: domestic supply; municipal uses—both ordinary and emergency; industrial use; the disposal of waste, both municipal and industrial; navigation; irrigation; electric power; recreation.

Water there must be for all these uses. But some of the uses conflict with use for public water supply. Consider the sources of water. It comes from rivers, lakes, wells, and distant storage reservoirs; and these water supplies change daily, even hourly, in terms of the weather or the acts or requirements of other users. Then there are the legal problems: waters that are intrastate, interstate, international, navigable, and non-navigable.

Yes, it is a complex world that lies behind the faucet, a world the housewife never thinks about. To an association that includes sanitary engineers, hydraulic engineers, plant operators, chemists, bacteriologists, and plant directors, I need hardly call attention to the challenge water supply holds for the professions.

To directors, administrators, and to lawyers I need hardly point out the challenge to democratic statesmanship. Indeed, the mechanical problems incident to the uses of water frequently pale before the human problems of professional training and social coordination, and to both of these the Federal Security Agency makes its contribution.

Oldest of the agencies which make up the Federal Security Agency is the United States Public Health Service. Many of you have been in active collaboration in its work, and its contributions to the sanitary control of drinking water are well known. I need mention only a few.

Drinking water on trains was a problem 30 years ago. Since the railroads passed through many towns and villages it was found that the variation of the water supplies multiplied the probability of infection. Travelers, who had become the victims of microbes

picked up at some remote water tower, became in their turn the focus of new epidemics in distant cities.

Recently the Public Health Service has reported on 52 cases of typhoid in several states from New York to California, all traced to a contaminated well at a midwestern cross-country bus junction. Thus, we are reminded of conditions which were not uncommon a few decades ago.

Drinking Water Standards

The first effort to meet this problem was the certification of drinking water required by the interstate quarantine regulations of 1912. This gave birth to the Treasury Standard for drinking water announced in 1914 and later superseded by the standards of 1925. These in their turn will give way within the next two years to new regulations which will embody a code of water works practice.

These standards are officially applicable only to the supplies from which interstate carriers draw their water, although that requirement applies to the nearly 2,000 points in the United States. Their original promulgation aroused protest in many of the communities which had formerly done a lucrative business with the carriers.

But as is usual in American technology, the people who cried, "It can't be done," were soon interrupted by people doing it. Community after community met the 1914 standards and planned supplies which would meet the standards of 1925. And these were not only communities which did business with common carriers but others as well. To the American public and the American municipal engineer these standards became the practical measure of a safe water supply.

The present standards, as you know, apply not merely to the actual water delivered but to the whole water-supply system—to the source, the reservoirs, the pipelines, pumping equipment, purification works, mains, and service pipes. They were not only a standard for water but a blueprint by which a sanitary engineer could show his community how to get and maintain a safe water supply.

These standards were not the work of some little group of specialists working remote from the problem in an isolated laboratory. In 1910, four years before their promulgation, the Public Health Service had detailed an officer to the study of stream pollution and its relation to the spread of typhoid fever. The facts that he gathered served as data for drafting the standards. In the task of formulating

these standards, the state health departments, the American Society of Bacteriologists, the American Chemical Society, the American Society of Railway Surgeons, the American Public Health Association, the American Medical Association, as well as your own American Water Works Association, participated. The standards represent the collective wisdom of those who had the job to do.

I would emphasize the cooperative formulation of the standards, for it epitomizes the relation which the Public Health Service has jealously maintained in its dealing with the states, with industry, and with local communities. It has served as an agency for fact finding, for consultation, for advice, and sometimes for emergency aid, not as a bureaucracy imposing its will upon those who should be its clients. It has let its professional prestige and the care with which it assembles its facts speak for themselves, for in the end they can speak much more loudly than can any arbitrary force.

After the formulation of adequate standards, there is still the question of conformity thereto. The problem of obtaining a safe water supply has become more difficult in the last 40 years, because of the increased pollution of possible sources. According to one report of the National Resources Committee, "On days of minimum flow about one quart of every gallon flowing in the [Ohio] River at some place has passed through a sewer system. This liquid after filtration is used for drinking purposes."

Federal Research

But how can the community insure itself a dependable water supply? How shall it obtain from the ever-changing water of adjacent streams or lakes safe water for its people? How can it know that the methods which it uses are up to date in their scientific efficiency and in their economy of the taxpayer's dollar? Cities cannot answer these questions for themselves. They demand research which is far too expensive for any single community to undertake on an adequate scale. They are being answered for communities by the Public Health Service.

For many years the Public Health Service has carried on research work seeking to improve the laboratory methods for water examination and to develop new methods for the purification of water supply. This has resulted in a more efficient operation of treatment processes and a more satisfactory control over the quality of the water delivered to the ultimate consumer.

You are all familiar, no doubt, with the work of the Public Health Service at its Cincinnati laboratory. Studies conducted there have been reported from time to time. Take one problem, for example. Water treatment devices like all other devices have their limitations and may become overloaded. How much pollution—bacterial, organic, and inorganic—can water contain and still be safely treated for municipal supply? Upon answers to such questions as that depend thousands of lives and millions of dollars of municipal investment. For answering just such questions the Service had, at the Cincinnati laboratory, a filtration plant complete in every detail and sufficient in capacity to supply a town of 2,000 people. Here water could be drawn from the Ohio River, from the city mains, and from the sewers of Cincinnati, creating any kind of mixture needed for test purposes.

In addition to this laboratory work conducted under conditions which closely approximate those of a municipality, extensive field work was conducted. Studies carried on in 17 river and 14 lake communities contributed to the result.

Under provisions of the Rivers and Harbors Act of 1937 the Service is cooperating with the United States Army Corps engineers to conduct a comprehensive survey of the Ohio River basin. This is one of the most thorough surveys yet undertaken. The objective of the survey is to determine the pollution existing, the remedial measures necessary for its abatement, the cost of such remedial measures, and the administrative procedures necessary for control.

Ohio River Compact

Nearly five years ago those of us who have been concerned with water supply and the problem of pollution in the Ohio Valley watershed drew up plans for an interstate compact, the Ohio River Compact, which was authorized by Congress in 1936. Six of the seven states originally concerned—Ohio, Indiana, Kentucky, Illinois, West Virginia, and New York—have already ratified this compact. Pennsylvania is expected to join and Virginia may also be added to the list. Senator Barkley has recently introduced a bill in the Senate which will automatically bring this compact into operation and set up a legally empowered interstate agency adequate to deal with the problem.

The Ohio River survey which I have described above will be completed in 1941. By the time the administrative organization of the

Ohio River Compact is complete, it will have at hand the basic facts necessary for the sound management of that great inland stream.

In reclamation areas and grazing districts many of the Civilian Conservation Corps projects have been concerned with the maintenance of a year-round flow of water for irrigation or for live stock. In other areas the conservation of soil and the prevention of floods have had their effect in reducing the silting of reservoirs or river beds. These boys work with the soil and you all understand the close relation between water use and soil use.

The work of the C. C. C. directly affecting the water supply of the Nation includes the cleaning and improvement of 21,200 springs, water holes, and small reservoirs. This work has been done in the seven years since the Corps was established in April, 1933. It also includes the building of 5,450,000 check dams primarily designed to prevent soil erosion, and more than 6,000 large impounding and diversion dams to check water flow during flood seasons, clearing and development of 179,000 acres of ponds and lake sites, construction of 28,700 water-control structures other than dams, building of 4,600 fish-rearing ponds, and the clearing and cleaning of 64,200,000 square yards of channels and levees.

I would prefer to discuss the contribution which the Civilian Conservation Corps makes to your enterprise in terms of men rather than of engineering, for the primary work of the C. C. C. is that of building men—not dams. Its objective is the conservation of opportunity, giving to boys, whose lives might otherwise have been injured by the depression, a chance to understand our country and its problems. They will be better citizens because of their chance for work and education.

The Federal Security Agency makes a direct contribution in personnel training for your work. Through the regional offices of the Public Health Service, consultation is available to states and to local communities (through their state health departments) on water supply as well as upon other problems of public health. The Service has from time to time assisted states in supervision and control of public water supplies by the detail of engineers.

These engineers have assisted in the training of state personnel and have brought about the establishment of engineering divisions in several of the state health departments. Service engineers from both the Washington and regional offices have lectured in the training courses conducted for water works personnel at various university

centers. For four years the stream-pollution laboratory at Cincinnati conducted training courses for state sanitary engineers. These courses may be resumed when the new laboratory is completed if there is evidence of a demand for such service.

Expanding Public Health Work

Under Title VI of the Social Security Act, grants-in-aid have gone to the states for the development of their public health activities, including their sanitary engineering services. The state engineering organizations during the last four years have increased their total personnel from 535 in 1936 to 825 in 1940. This increase includes the especially significant addition of 166 sanitary engineers to the state departments. There were 187 engineers employed by the states in 1936. Today there are 353. Total budgets for sanitary engineering have increased by $1,000,000, more than $400,000 of which came from the states themselves. It is not possible to say how many of these engineers are employed in water service work, but this increase cannot help but mark a step forward in your services.

Vocational Education

Under the George-Deen Act, passed by Congress in 1936, Congress authorized state boards for vocational education to utilize Federal grants-in-aid for the vocational education of public employees. Under this provision the Federal Security Agency has found it possible—through the Office of Education—to cooperate with a few of the state boards for vocational education in training programs for water works operators. Four state boards for vocational education—Michigan, New York, Oklahoma, and Texas—conducted training programs for more than 1,000 water works operators in the year ended June 30, 1939. A fifth state, Louisiana, has just initiated a similar program.

And for those who look for the logic in Federal reorganization, I offer these twin contributions of the Public Health Service and the Office of Education as "Exhibit A." For whereas the Public Health Service had sometimes provided lecturers or instructors for these courses, it is noteworthy that some of these were not aware that Federal funds from other sources had contributed to the training expenses of the students. And those who had organized and financed the courses for the students sometimes were not aware that other Federal agencies were contributing to the teaching staff.

With these interests unified and coordinated, we move forward to a period in which more effective training can be extended, perhaps to a time when you can promulgate professional standards for the personnel of your field and the guidance of future training programs.

Emergency Services

Finally, there is always the chance of emergency. In 1937 my own valley, the Ohio, was visited by one of the most disastrous floods in its history. Before the rising waters of the Appalachian streams had reached Pittsburgh's "Golden Triangle," the health officers of every state of the Ohio valley had conferred by telephone with Surgeon General Parran. Supplies to fight the epidemics that may be the aftermath of any flood were gathered together. Every available officer of the Service was mobilized and dispatched to appropriate points in the threatened states to assist the state health department personnel. Forty-five sanitary engineers from 10 states were loaned—volunteers under Public Health Service direction for this war the waters waged against the peaceful peoples of the river towns.

There were many problems that faced the quickly mobilized public-health army. There were medical and nursing needs. There were needs for immunization against typhoid and diphtheria and smallpox. As the waters receded, there was need for the rehabilitation of safe milk and food supplies. But most critical of all was the need of those towns for water. Without safe water no family could safely stay in its home. No family that had fled could return until safe water was available.

A little map at staff headquarters in Washington told the story of that fight most dramatically—a little map on the wall of the office of the Public Health Engineering Section.

That map showed the water supplies clustered along the river. There were yellow pins for filtered supplies which were out. There were red pins for well supplies out. There were green pins for water supplies back in service. Just before the first of February of 1937 the water supplies for 1,600,000 persons along that river were out.

During the days that followed, tank cars were loaned; transportation was donated by the railroads; water was shipped from Chicago, from Columbus, from hundreds of other towns. Sanitary engineers supervised hundreds of volunteer workers in the rehabilitation of municipal supplies. They checked the water distribution of homes;

they drained home cisterns, chlorinated them, and refilled them with filtered water; they cleaned up private wells; they cleaned up the sewage disposal systems and put them again into effective operation.

It was a two-week fight, but the fight was won. By February 12, the water supplies for more than 1,400,000 of these refugees from the river were again in operation. Again they could go back and take for granted the water which so magically flows from the kitchen faucet. But for two weeks there had been dramatized the age-old fact that the community that fights for water fights for life.

It is not often that the problem of water supply is so dramatized. Pollution is seldom so quickly fatal or so readily apparent. The need for skilled men or the need for a sane and selfless cooperation is seldom so apparent, but the need is always there, and our cooperation is always the same. Whether the need be sudden or merely the less dramatic but long-run need, the Federal Security Agency operates on the basis of professional cooperation and collaboration.

Changes in Patent Laws

By Arthur C. Brown

MANY of you no doubt, from time to time within the past few years, have read of bills introduced, or proposed for introduction, to Congress for changing our patent laws. The author must confess to a sense of relief upon comparing the amendments actually passed by the 76th Congress with some of those proposed, but which fortunately failed of passage.

We must not overlook the fact that, following the rule of the greater good to the greater number, patents are primarily granted for the benefit of the public, not the inventor, and that any law tending to impose limitation of the patent monopoly to its legal period is salutary.

Article I, Section 8, Clause 8, of the Constitution, from which Congress derived its authority for such legislation, reads as follows:

"The Congress shall have power . . . to promote the progress of science and useful arts, by securing for limited times to authors and inventors the exclusive right to their respective writings and discoveries."

That article provides an excellent example of the clarity of expression employed in preparation of that great document.

It left no doubt of the intent of the framers that the privilege of patent monopoly was to be granted, not as a special favor, but to those advancing the arts and sciences by adding to the fund of human knowledge. The purpose was to limit the duration of such monopoly to periods, which, sufficient to enable authors and inventors to reap a just reward from their expenditures of time and money in development of their writings and inventions, would, nevertheless, pass the benefits to be derived from such writings and inventions on to the public at the earliest date consistent with fair dealing.

A paper presented on April 25, 1940, at the Kansas City Convention by Arthur C. Brown, of the Kansas City, Missouri, Bar.

While the purpose of the Constitution was clear in that regard, its enactment into statutory law was fraught with difficulty.

The First Patent Act

The Patent Act passed by the First Congress in 1790 provided for granting a patent for an invention which was "not before known or used," but it did not state whether the knowledge or use to defeat a patent must have been before the date of the invention, or before the date of application for patent for it. The latter construction was given the Act, and as no time was fixed during which the invention may not have been in use, patents were frequently granted for inventions which had been in public use by the inventors for many years prior to their applications for patents on them.

In an attempt to correct this situation the Act of 1793 was passed. It provided for granting a patent for an invention not known before the *date of application*, but included as a defense to a suit for its infringement, public use or publication of the invention prior to the patentee's discovery of it.

Such intended construction of the Act led to much confusion, due to different interpretations given it in the courts. It was not until 1836 that the Act of 1793 was amended to provide for grant of patents for inventions not in public use, *or on sale* by others, with the consent of the inventor, before his application therefor; or if the invention had not been previously made by any other person in this country, or patented or described in this or any foreign country prior to date of the application. No public use or sale of the invention by others, without the consent of the inventor, was a bar to the patent.

By the amendment of 1839 a public use, but not sale, for more than two years prior to the date of application, was held to deprive an applicant of the right to a patent.

Substantially the same provisions were carried into the Act of 1870, which became Section 4886 of the Revised Statutes of 1874, and it was not until passage of the Act of 1897 that the two-year rule was made to apply to sale and publication as well as public use of the invention. At that time the bar of public use or sale was made to apply to public use or sale only in this country.

Section 4887 of the Revised Statutes substantially repeats the provisions of Section 4886, except to provide that when a patent has first been issued in a country foreign to the United States, an application for patent for it in this country must be filed within twelve

months, or in case of a design patent, within six months, of the date of the foreign application. This section also gives the inventor the benefit in this country of the date of first filing of the application in a foreign country, provided that country, by treaty, gives the same privilege to citizens of the United States.

Section 4929 is the section of the Revised Statutes relating specifically to issuance of design patents, and contains the same restrictions as to prior patenting, publication, prior public use and sale found in Section 4886 just discussed.

Section 4920 is the Section of the Revised Statutes making such prior patenting, publication, prior public use and sale of patented inventions special defenses in patent infringement suits.

Time Allowed for Application

Public Act No. 286, passed by the recent Congress and approved August 5, 1939, merely changes Sections 4886, 4887, 4920 and 4929 to reduce the period of allowable publication, public use or sale of an invention prior to date of application for a patent, from two years to one year. The Act does not go into effect until August 5, 1940—one year following its approval, thus avoiding a retroactive operation.

A patent, except for a design, is granted for a term of seventeen years, which runs from the date of its grant—not from that of its application, but notice that application for patent on a device or process is pending deters others from engaging in competition with the applicant on the patented article while the application for it is pending. Thus the owner of an application for patent, which finally eventuates into a patent, may enjoy the benefit, not only for the seventeen year rightful monopoly afforded by his patent, but also for the additional term during which the application for it was pending.

The author wishes to make it clear that he is not saying that the time allowed for legitimate prosecution of an application for patent constitutes a wrongful extension of the rightful monopoly. Time must be allowed for proper prosecution of every application, but this privilege can be, and occasionally is, abused. When the application is filed in the Patent Office, it may contain errors requiring correction, or claims that must be narrowed or otherwise revised for protection of the public, as well as of the applicant, and a reasonable time must be allowed the applicant for effecting such corrections and revisions.

Time Limit for Response to Patent Office Actions

When the Patent Office was first established, no time for response to actions on applications was set, and it was not unusual for them to remain pending for as long as ten or more years before being passed to issue.

This practice naturally led to abuses, but it was not until 1870 that an Act was passed limiting the time for responses to rejections and other Office actions. Then the period was limited to two years, and the penalty of abandonment imposed for failure to respond within the period. This was written into Section 4894 of the revised statutes, but by the Act of 1897 the time was reduced to one year, and in 1927 to six months, where it now remains.

Public Act No. 341, approved August 7, 1939, left the basic period for responses to Patent Office actions at six months, but gives the Commissioner of Patents authority to reduce it to a shorter time, not less than thirty days, by written order to the applicant, thus speeding prosecution of applications that have been pending for more than three years, or for other good cause. The Act went into operation upon its approval.

It is the first inventor of patentable subject matter who is entitled to the patent on it. This is true even though another has actually secured the patent before the first inventor files his application. Under such conditions the actual first inventor, encountering a patent issued to another for this invention, copies the claims from the issued patent and asks that his pending application be placed in interference with the issued patent, so that, upon submitting proof of his invention prior to that of the patentee, he may be awarded the interfering claims.

Formerly the only restriction to such procedure imposed upon the applicant was that he be able to prove a date of invention prior to that claimed by the patentee, and that his application be filed within the statutory period of two years following the actual issuance of the other patent.

To conform such cases to the recent Acts reducing the statutory period of public sale, use, patenting and publication, Public Act 288, approved August 5, 1939, provided that no amendment to pending applications, in which claims of an issued patent are made for the purpose of interference, may be filed more than one year following date of grant of the prior patent. To avoid conflict with the two-year period within which such amendments might formerly have been

made, the Act was written so it would not go into effect until one year after its approval.

Changes in the Term for Patents

Pursuant to the Act of 1790 patents were issued for a term not to exceed fourteen years.

The Act of 1836 limited the term to fourteen years, and fixed the filing fee at $30, of which the applicant was entitled to a return of $20, if he chose to abandon or withdraw his application.

But by the Act of 1836 patents were made renewable for an additional term of seven years upon showing that the patentee had not been adequately rewarded for his contribution to the welfare of the country, and upon payment of an additional fee.

By the amendment of 1861 this was, however, changed to provide a single term of seventeen years. A filing fee was required upon filing of the original application. A further fee was required upon issuance of the patent, but no time was specified within which the second fee should be paid, thus permitting an applicant to defer issuance of his patent indefinitely.

This was corrected by the Act of 1863, which required payment of the final fee within six months after the application was allowed, and that the patent be forfeited if the final fee were not paid within that period.

By the Act of 1865 applications allowed and forfeited could be refiled within two years after the date of allowance of the original application.

The Consolidated Patent Act of 1870, and the Revised Statutes of 1874, contained the same provisions, but by the Act of March 2, 1927, the period of two years allowed for renewal was reduced to one year from the date of allowance of the application. The applicant seeking renewal was required to pay a new filing fee with the application for renewal in addition to the issuing fee.

Except for relatively minor changes the law of renewals remained the same until the passage of Public Act No. 358, approved on August 9, 1939. This Act abolishes renewals, but gives the Commissioner of Patents authority, in his discretion, to receive the final fee, if paid within one year after the six months period following notice of allowance of the application, upon payment of an additional fee of $10. This Act took effect upon approval, but does not apply to applications in which the notice of allowance had been previously issued.

In other words, an application, forfeited for failure to pay the final government fee, cannot now be renewed, but a patent may be granted upon the original application upon payment of the final government fee within six months after allowance, or within an additional one year, upon satisfactory showing to the Commissioner of Patents that the delay and payment of the final fee were excusable for all or any part of that time.

Jurisdiction over Appeals

The Act of 1870 established the position of a Special Examiner of Interferences, whose decisions in interference cases were appealable to the Commissioner of Patents, and made decisions of the Commissioner appealable to the Supreme Court of the District of Columbia, sitting in *banc*.

In 1891 jurisdiction of appeals from the Commissioner of Patents was transferred to the newly organized Court of Appeals of the District of Columbia, where it remained until transferred to the Court of Customs and Patent Appeals, which was organized in 1929, partly to take over such jurisdiction.

Section 1 of Public Act No. 287, approved August 5, 1939, merely substitutes a board of three examiners of interferences for the single examiner for hearing interference cases in the first instance, and abolishes appeals from such board to the Board of Appeals in the Patent Office, thus simplifying procedure by elimination of one appeal in such cases.

Section 2 of the Act merely conforms Revised Statutes Section 4909 to 4904 by elimination of reference to appeals in interference cases from the Examiner of Interferences to the Board of Appeals.

By the amendment of 1861, twice rejected applications and decisions of a primary examiner in interferences between pending applications, or a pending application and an issued patent, were made appealable to a Board of Appeals in the Patent Office, decisions of which were appealable to the Commissioner of Patents.

The Act of 1893 created the Court of Appeals of the District of Columbia, which was authorized to hear appeals direct from the Commissioner of Patents.

The amendment of 1927 provided for appeals from the Board of Appeals to the Court of Appeals for the District of Columbia instead of to the Commissioner of Patents, and gave the appellant the choice of appeal from the Board to the Court of Appeals or proceeding by

bill in equity to the Supreme Court of the District of Columbia, and, finally, appealing from its decision to the Court of Appeals of the District of Columbia.

In 1929 appeal from the Commissioner to the Court of Appeals was transferred to the newly organized court of Customs and Patent Appeals, but the right to proceed by bill in equity to the Supreme Court of the District, or to appeal from the decision of the Commissioner of Patents to the Court of Appeals of the District, was retained.

Sections 3 and 4 of Public Act No. 287 (1939) merely amended Section 4911 of the Revised Statutes to provide that the decision, sought to be corrected in interference cases, must have been given by the Board of Interference Examiners instead of the Examiner of Interferences, and to strike out, in connection with interference cases, reference to the Board of Appeals as the reviewing tribunal, except for cancellation of reference to certain procedural requirements contained in the last paragraph.

Section 4 of the recent Act merely clarifies the language of Revised Statutes Section 4915 regarding procedure by bill in equity to review decisions of the Commissioner of Patents in *ex parte* cases, adds a similar procedure in interference cases, and cancels certain paragraphs of the statute relating to procedure in such cases.

The Act went into effect two months after its approval on August 5, 1939, except as to interferences then pending, regarding which the former proceedings were retained.

Another act passed by the recent Congress having to do with administration of the Patent Office, rather than with any substantive rights, is Public Act No. 244, approved July 31, 1939, transferring jurisdiction over the registration of prints and labels from the Patent Office to the Register of Copyrights, where it has always rightfully belonged. This Act takes effect on July 1, 1940. The transfer is to be made on June 30, 1940, by returning all pending applications, fees and deposits in connection with pending applications for such registrations to the applicants, so that the applications may be refiled with the Register of Copyrights in the Library of Congress.

Not all of the legislation relating to patents proposed for passage by the last Congress was enacted into law.

A bill to provide a permanent force to classify patents, known as House Rule 6721, was passed by the House of Representatives on July 6, 1939, but did not reach the Senate. As this bill, like those

actually passed by both the House and Senate, relates to simplification of administration of the Patent Office, it will no doubt receive favorable consideration when it reaches the Senate.

Another bill, No. 2688, directed toward change of the term of a patent from seventeen years following date of its grant to twenty years from the date of filing of the application, was introduced into the Senate and favorably reported upon by the Senate Patent Committee on July 11, 1939. It will no doubt come up for consideration at a later session of Congress.

Another bill directed toward creation of a single Court of Appeals for patent suits was introduced into the Senate as No. 2687, and was favorably reported by the Senate Patent Committee, but not reached for consideration. Passage of this bill is doubtful, however, as the same legislation has been proposed but rejected by many previous Congresses.

This is the extent to which the Patent Law was amended by the 76th Congress. From even such a brief consideration of the changes made, it is apparent that they involve no radical tampering with the patent laws, but, rather, are mere simplifications of procedure in the securing of patents. A salutary reduction is provided in the time permitted in prosecution of applications for patents in the Patent Office and of the periods of the patent monopoly following first knowledge of an invention by the public through its public use, sale, or description in a patent or other publication. The intent of the framers of the basic law that the public should be accorded the benefits of new discoveries at the earliest possible date consistent with encouragement to those willing to spend their personal time and money in the conception and development of inventions is thus accomplished.

Discussion by *D. L. Erickson.* * Contrary to the experience of engineers in other fields of practice, the water works engineer is not ordinarily concerned with patents or patent infringement. This is particularly true with respect to water purification processes. One reason for this fact may be that a large majority of the water works engineers are public employees, who are primarily engaged in producing water for the public that is safe from a health standpoint. Usu-

* Director, Dept. of Parks, Public Property and Improvements, Lincoln, Neb.

ally, when a water works engineer does develop a new process in water purification or improve an old process, he immediately describes it in some engineering publication and freely gives the profession the benefit of his work. The JOURNAL is full of such material.

The water works engineer is, therefore, usually public-minded rather than patent-conscious. He is ordinarily not familiar with patent law, and usually does not know how to protect his own work, based on his own research and findings, against more patent-conscious persons who may have observed his work and secured a patent based on his developments.

When the water works engineer has his first personal experience with patent law, he will probably be quite surprised. He will first learn that patent law is such a specialized branch of law, with which the ordinary law practitioner is not familiar, that it becomes necessary to employ an expert patent lawyer. The result is that patent litigation is quite expensive. He will then learn one of the fundamental provisions of patent law, which, in substance, is that the holder of a patent is protected for a period of two years prior to the date of application for the patent, so far as prior use is concerned. In other words, in order to rely on an example of prior use as an argument for invalidating a patent, such prior use must be more than two years before the date of filing the patent application. Of course, a patentee must under the statute be the *first* inventor. Therefore, any knowledge or use of the invention by any one before the patentee made his invention would invalidate the patent. This would be true regardless of whether such knowledge or use was acquired less than two years before the filing of the application, as long as it was prior to the date of the invention by the patentee.

The City of Lincoln recently had a case in patent litigation which may be of interest to water works engineers. The case involved the iron and manganese removal process used in its plant at Ashland, Nebraska (Jour. A. W. W. A. **27**: 337 and **29**: 1896). The process involved pre-chlorination, aeration, chlorination, contact filtration, sedimentation and rapid sand filtration.

The main events in connection with the plant may be summarized chronologically as follows:

1. September, 1933. Discovery of manganese in water.
2. October, 1933. Employment of John R. Baylis as consulting chemist to help solve the manganese problem.

3. January, 1934. Construction of an experimental filter by the City at Ashland, according to the suggestion of Mr. Baylis.
4. January, 1934. Employment of the firm of Black and Veatch as engineering consultants.
5. May 14, 1934. Official filing of plans of iron and manganese removal plant by Black and Veatch.
6. September 17, 1934. Award of contract for construction of the iron and manganese removal plant.
7. November 8, 1934. Paper by the city engineer describing experimental filter and design data read at the Missouri Valley Section Meeting of the A.W.W.A.
8. October 9, 1935. Plant put into operation.
9. June, 1937. Paper describing design, construction and operation of plant read at the Buffalo Convention of the A.W.W.A.

On February 15, 1939, without any previous knowledge of an application for a patent upon such a process ever having been filed, the City was notified by a local attorney that his client had, on January 17, 1939, been granted a patent on this process, that Lincoln's Ashland Plant was an infringement on his patent and that his client wanted either a lump sum of $12,500 or $1,500 per annum for twenty years.

This patent had been granted to a certain local chemist connected with another public agency and who, for several years, had made chemical analyses for the Lincoln Water Department on an individual sample basis. The City contends, however, that, after the employment of Mr. Baylis and Black and Veatch, this chemist had no connection with the experimental filter conducted by the City under the direction of a full-time city chemist, nor with the design of a final plant.

The lawyer for the chemist maintained that his client had conducted independent research work in his laboratory on which his alleged invention was based. In this connection, the City disclaims knowledge of his having done any independent research work relating to iron and manganese removal.

According to the United States Patent Office, this Patent No. 2,144,051, was applied for on October 29, 1935.

Attention is called to this date of filing the patent application and

the fact that this date was after the completion and commencement of operation of the Ashland Plant, and nearly a year after the reading of a paper at the Missouri Valley Section Meeting of the A.W.W.A., describing the experimental filter and design data from which the final plant was built.

One of the technical articles most useful to the City when it was conducting its experimental research work on iron and manganese removal was an article published November 29, 1933 in *Waterworks Engineering* describing the use of pre-chlorination in iron and manganese removal at Wellsburg, West Virginia. This article was specifically referred to in the report of Black and Veatch. It was later learned that the information contained in the article was read as a paper before the West Virginia Eighth Annual Conference on Water Purification held on October 18, 20, and 21, 1933. It is to be noted that this date is slightly more than two years before the filing of the application for the patent herein referred to. The Wellsburg process has an important bearing on this case, because of the similarity between it and both the Lincoln process and the process described in the patent.

The general purpose of the alleged invention covered by the patent is stated in the specifications as follows:

"This invention relates to the treatment of water for the purpose of removing certain impurities contained therein, such as salts of iron and manganese, and especially salts containing ferrous and manganous ions."

The particular process described in the patent in considerable detail is set out in the following statement:

"In carrying out the process, the water is first aerated so as to increase its pH value and render it slightly alkaline. It is then treated with small quantities of chlorine, and passed through a contact bed, after which it is permitted to coagulate by standing and is finally filtered through ordinary sand or other similar filters."

The patent contains six claims on processes for treating water containing iron and manganese which may be summarized as follows:

1. Increasing the pH to 7.0, chlorination, and removing precipitate.
2. Increasing the pH to 7.0 by aeration, then chlorination and removing the precipitate.

3. Raising the pH to an alkaline condition, oxidation by chlorination, contact bed, and removing the precipitate.
4. Raising the pH to an alkaline condition, oxidation by chlorination, use of a catalyst, coagulation and settling, and filtration.
5. Raising the pH to an alkaline condition, oxidation by chlorination, passage through a porous substance and securing catalytic precipitation, coagulation and settling, removing the settled material and filtration.
6. Raising the pH to an alkaline condition, oxidation by chlorination, use of a catalyst, coagulation, removal of precipitate and coagulate by sedimentation and filtration.

The patent claims do not specify any particular quantity of chlorine which is to be applied to the water. The patent specification, however, mentions an exceedingly small amount of chlorine which averages about .25 and seldom exceeds 0.5 part per million.

In the Lincoln plant, 2.0 p.p.m. of chlorine are applied and this raises the question as to whether this plant was an infringement of the patent.

The patent attorney retained by the City was of the opinion that this patent was invalid for the following reasons:

1. Certain foreign patents already cover the use of chlorine and contact beds in iron and manganese removal.
2. It was old in the art to adjust the pH value of water to a slightly alkaline condition.
3. The essential steps of the alleged new process were all old, as evidenced by articles by Weston, Baylis, Wolman and others in various technical publications.
4. The patent was completely anticipated by the reading of the paper describing the Wellsburg, West Virginia, process and by the further fact that this process was put in use some two and one-half years before the filing of the patent application.

In June, 1939, the holder of the patent filed with the City Clerk a notice of intent to sue the City of Lincoln for patent infringement. Several months elapsed with no suit being filed. The City was desirous of having the matter adjudicated as soon as possible while those responsible for the design of the Ashland Plant were available, so on November 13, 1939, the City filed an action in Federal Court asking that the patent be declared invalid.

Immediately there commenced a long series of negotiations between the attorneys representing the City and the holder of the patent, and on March 18, 1940, the case was settled by both parties entering into a stipulation under which the City recognizes the existence of the patent but does not admit its validity, denies that the owner of the patent was the inventor of the process and denies that the City of Lincoln was infringing said patent. Under the stipulation, the City furthermore agreed to pay the owner of the patent the sum of $750, in consideration of which the City received a license to use the patent process. In recommending this settlement to the City Council, the City Attorney stated that the prosecution of the City's suit for the cancellation of the patent would involve further expenditure of some two or three thousand dollars.

The City was confident that final adjudication of this suit would have resulted in the patent being declared invalid, but since it had already expended some $1,800 on this case and could save money by settling it for $750, the City Council decided that settlement on the above basis was for the best interest of the City.

Fractionated Coagulation—A New Procedure

By Rogelio A. Trelles, Daniel J. Bengolea And Adolfo Gonzalez Poccard

ABOUT 25,000 tons of alum are used yearly in the purification of the River Plata water to supply the City of Buenos Aires. This water can be coagulated readily. Research was initiated to find a method to reduce the consumption of alum, and to improve the quality of the settled water.

The River Plata water does not possess constant physico-chemical features and varies even at very brief intervals, especially in turbidity, alkalinity and pH. The low and high tides of the river and the rainfall, are the most important causes of these variations. Table 1 shows some characteristics of this water.

Turbidity is totally due to clayish earth, as an actual suspension settling mostly by plain sedimentation. A previous sedimentation has no influence whatsoever upon the dose of coagulant; on the contrary, its effect is to give a floc of less density.

It was established by the centrifugal tests carried out at 3,000 and 25,000 r.p.m., that turbidity is an actual suspension, not colloidal. In the first case a slight opaline water was obtained, and in the second one, viz. 25,000 r.p.m., a perfectly clear and sparkling water.

The color, due to organic matter of vegetal origin, seemed to be partly colloidal and partly actual solution, since it was not possible to eliminate it entirely by coagulation.

Up to the moment of starting the fractionated coagulation experiments, little was known about the different factors which influenced the coagulation phenomenon of the River Plata water. It may now be stated that it is possible to enlarge the range of the optimum coagulation pH, reduce the dose and shorten the sedimentation time, either by fractionating the coagulant dose or by dispersion.

A record of research contributed by Drs. Rogelio A. Trelles, Daniel J. Bengolea and Adolfo Gonzalez Poccard, Departamento Tecnico, Obras Sanitarias de la Nacion, Buenos Aires, Republic of Argentina.

Fractionated coagulation consists of adding the total dose of coagulant in two fractions, leaving a time interval between doses, and employing either aluminum salts only, or using them for the first fraction and iron salts for the second one.

This coagulation procedure, conceived by Dr. Rogelio A. Trelles, has given excellent results with the River Plata water. Following is a summary of the experiments carried out with this procedure.

Materials Used

Diffuser. The diffusing device used makes possible variable speeds between 180 and 280 r.p.m. The stirring paddle is of rectangular shape and measures 90 x 30 mm. For all tests, a speed of 250 r.p.m. was used.

TABLE 1

Characteristics of River Plata Water

(Determinations According to *Standard Methods*)

	MINIMUM	MAXIMUM
Color (p.p.m.)	12	50
Turbidity (p.p.m.)	25	500
pH	7.1	8.2
Hardness (p.p.m.)	20	60
Alkalinity (p.p.m.)	25	100
Dissolved solids (p.p.m.)	100	300

Stirring Apparatus (Flocculator). The flocculator consists of six vertical shafts, each with a transversal paddle 90 x 20 mm. The speeds can be regulated between 10 and 100 r.p.m. These tests were run at 70 r.p.m.

Containers. One-liter beakers, 10 cm. in diameter.

Reagents. One per cent alum solution, one per cent chlorinated copperas solution, and one per cent ferric chloride solution.

Turbidimeters. For turbidities above 25 p.p.m., the Jackson turbidimeter was used. Turbidities below 25 p.p.m. were determined by comparison with silica types, in colorless glass containers, following the instructions given by *Standard Methods.*

1. Comparison between the unfractionated and fractionated coagulation: On the basis of the previously determined optimum alum dose, the water obtained was compared with the one treated with a similar dose, but added in a fractionated manner. The results obtained are shown in Figs. 1 and 2.

2. Comparison of fractionated alum: Comparative tests employing fractionated alum, alum plus ferric chloride and alum plus chlori-

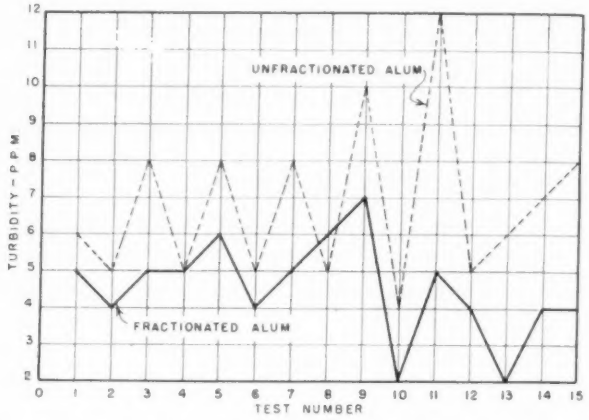

FIG. 1. Turbidity of Water after Treatment with Alum, fractionated and unfractionated

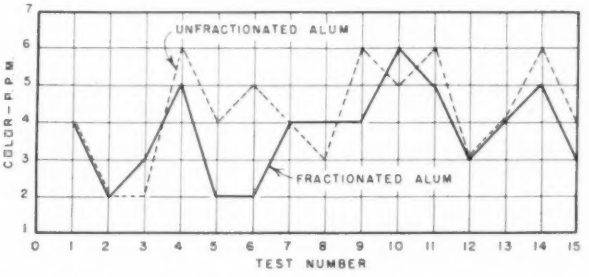

FIG. 2. Color of Water after Treatment with Alum, fractionated and unfractionated

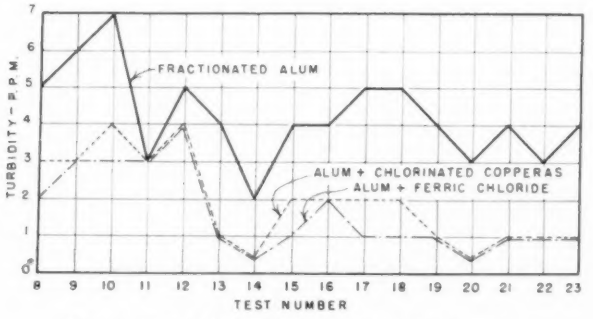

FIG. 3. Turbidity of Water after Treatment

nated copperas, gave the results which are shown in Figs. 3 and 4. For the tests, a one-minute interval was used between each addition.

3. Fractionation of the coagulant dose: In order to establish how the dose should be fractionated, and to obtain in the meantime the relation of aluminum salt to iron salt, a selection was made among those tests which gave waters of lowest turbidity and color. As shown in Table 2, the relation of alum to the total dose varied from 58 to 71 per cent; therefore, the corresponding relation of iron-salt lies between 29 and 42 per cent. On this basis the dosage used in subsequent tests was fixed at $\frac{2}{3}$ alum and $\frac{1}{3}$ iron salt.

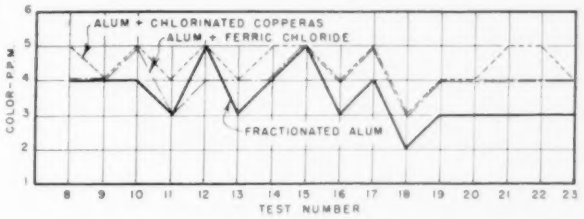

FIG. 4. Color of Water after Treatment

TABLE 2

Relation of Aluminum Salt to Iron Salt in Total Dose

TEST NO.	TOTAL DOSE		PER CENT RELATION OF ALUM TO TOTAL DOSE	TUBIDITY	COLOR
	$(SO_4)_3Al_2$	Cl_3Fe_2			
				p.p.m.	*p.p.m.*
2	30	20	60	3	2
3	30	15	66	2	3
10	50	20	71	1	3
14	45	20	69	0.5	4
16	30	15	66	2	4
18	30	15	66	1	3
19	40	20	66	1	4
20	35	20	64	0.5	4
23	35	25	58	1	4

During all these comparative tests, between the fractionated and unfractionated coagulation, the coagulant was diffused in the first case, after the aggregate of each fraction; on the other hand, the diffusion was not utilized when the unfractionated dose was added. (Method used to determine the dose at the Palermo Waterworks.)

Since this was the only existing difference between the operating methods followed for these comparative tests, because of the importance which the coagulant diffusion could have, it was decided to start another series of tests in order to determine the grade of influ-

ence of the diffusion, as well as the different periods of agitation (flocculation).

4. Influence of the flocculation: For the flocculation tests the alum dose was adjusted in such a manner that water agitated during 10 minutes would show an acceptable turbidity. The turbidity was valued after 30 minutes of settling in the glass containers. As may

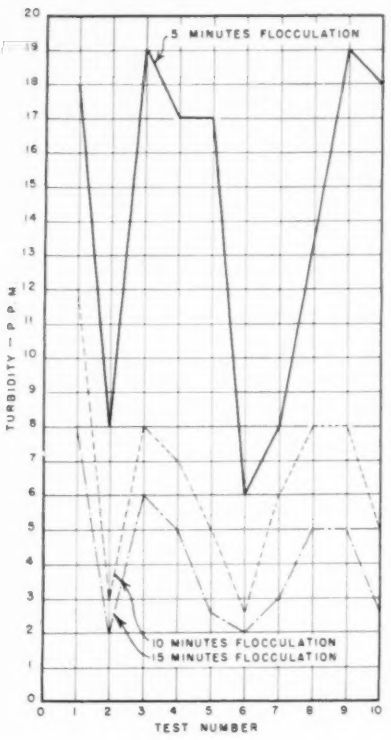

FIG. 5

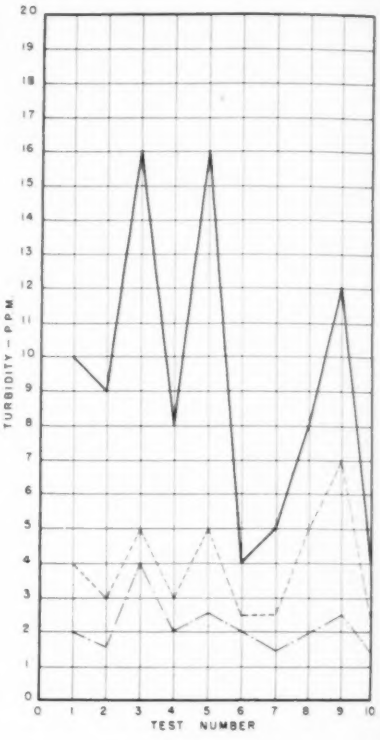

FIG. 6

FIG. 5. Influence of Flocculation, without previous diffusion
FIG. 6. Influence of Flocculation Time, using previous 5-second diffusion

be seen in Fig. 5, the turbidity decreases as the flocculation time increases.

5. Influence of diffusion: In order to demonstrate the influence exerted upon the flocculation by the previous diffusion of the coagulant, experiment 4 was repeated, but in each case the coagulant was diffused during 5, 10 and 20 seconds. As shown in Figs. 6, 7 and 8, it may be stated that a 10-second diffusion has more influence than a 5-second one, when the flocculation time is 5 min. For 10- and

15-minute flocculation, the difference is less and in a few cases the increase in diffusion time was not favorable. Finally, a diffusion of more than 10 sec. was found to be sometimes inadvisable in that no advantages were obtained to warrant the greater diffusion.

6. Comparison of single and double diffusion: Since it is necessary, while effecting the fractionated coagulation, to diffuse the coagulant twice (once after each fraction of the dose), it was desirable to ascertain whether this double diffusion would have an influence upon the results. For this purpose, 10-second diffusion tests were carried out,

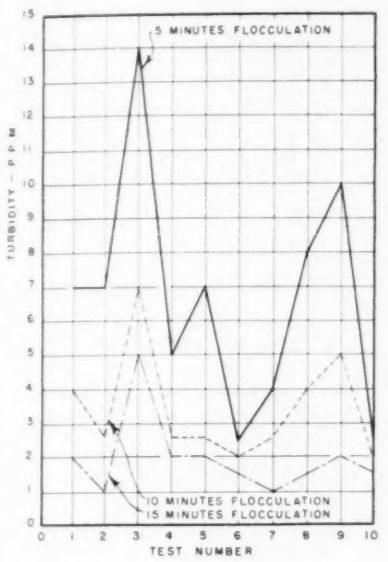

FIG. 7

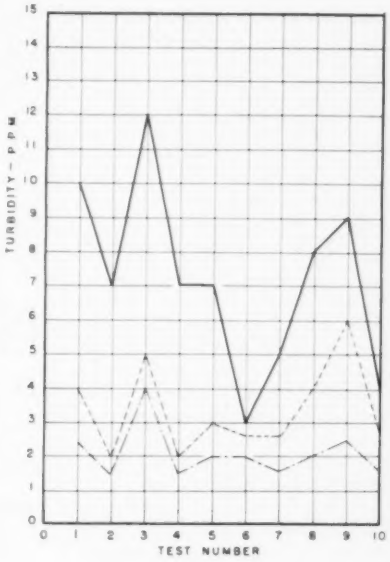

FIG. 8

FIG. 7. Influence of Flocculation Time, using previous 10-second diffusion
FIG. 8. Influence of Flocculation Time, using previous 20-second diffusion

and these were compared with others in which the diffusion was effected twice, with a one-minute interval between each one, i.e. in a manner similar to the fractionated coagulation method.

Table 3 shows that the same results are obtained with a 10-second single diffusion as with a double one of 5 + 5 seconds. From this fact it was inferred that the benefits of the fractionated coagulation are not due to double diffusion.

7. Determination of the most convenient time interval between the addition of the two fractions of the coagulant dose: The tests to establish the most convenient time interval between the addition of the

fractions, are demonstrated in Table 4. It is evident that this time lies between 90 and 120 sec.

8. Influence of the activated sodium silicate: Knowing the influence exerted upon the floc sedimentation by the activated sodium silicate previously added to the coagulant (Baylis Method), it was decided to use it also with the fractionated coagulation. The tests were

TABLE 3

Comparison of Turbidity after Single and Double Diffusion

TEST NO.	TURBIDITIES IN P.P.M. (STANDARD METHODS) DIFFUSION TIME		ULTIMATE pH
	10 Seconds	5 + 5 Seconds	
1	3	3	6.5
2	3	3	6.5
3	3	3	6.5
4	2.5	2.5	6.5
5	3	3	6.6
6	3	3	6.6
7	4	4	6.5
8	4	4	6.6
9	3	3	6.7
10	2.5	3	6.5

TABLE 4

Results of Tests to Determine Best Time Interval in Fractionate Dosage

TIME BETWEEN EACH ADDITION	WATER TREATED WITH FRACTIONATED ALUM	
	Turbidity	Color
sec.	*p.p.m.*	*p.p.m.*
30	5	4
45	4	4
60	4	3
90	3	2
120	3	2
180	4	3
240	4	3

carried out using alum in the first fraction of the dose, and ferric chloride in the second one, with and without previous addition of silicate. The results are shown in Table 5.

9. Influence of fractionated coagulation on the doses of coagulant: Ten comparative tests were carried out, following three methods, with 5-minute flocculation for all of them: first, coagulation without

diffusion; then, coagulation with fractionated diffusion of 5 + 5 seconds; and finally, fractionated coagulation. The dose of coagulant was regulated so that the settled waters would show analogous

TABLE 5

Influence of Activated Sodium Silicate on Turbidity and Color

(Results in p.p.m.)

TEST NO.	TURBIDITY			COLOR		
	Raw water	With silicate	Without silicate	Raw water	With silicate	Without silicate
1	90	2	5	22	2	2
2	95	1	4	18	3	3
3	110	2	6	18	4	5
4	100	2	6	16	3	3
5	140	3	6	22	3	3
6	150	2	5	18	3	4
7	150	1.5	5	14	2	3
8	170	0.5	3	17	3	4
9	138	0.5	3	14	4	5

TABLE 6

Influence of Fractionated Coagulation on Alum Dose

(Results in p.p.m.)

TEST NO.	DOSE OF ALUM			TUBIDITY		
	a*	b†	c‡	a*	b†	c‡
1	50	45	40	3	2.5	2.5
2	45	45	40	2	2	1.5
3	55	50	45	3	2	2
4	60	55	50	2.5	2	1.5
5	90	80	75	3	3	2.5
6	85	80	75	2	2.5	2
7	100	90	85	2.5	2	2.5
8	60	60	55	3	3	2.5
9	55	55	50	2.5	2.5	3
10	70	60	55	3	3	2.5

* Coagulation without diffusion.

† Coagulation with fractionated diffusion of 5 + 5 seconds.

‡ Fractionated coagulation.

turbidities. The results are shown in Table 6. From it may be drawn the conclusion that 8 per cent can be saved by diffusion of coagulant, and 16.7 per cent by fractionated coagulation.

Aside from these tests, and in order to discover whether good results could also be obtained by modifying the sequence of the additions of the different coagulants, as well as by altering the relations between them, other tests were effected with:

1. Mixture of both coagulants, alum with iron salt, before adding it to the water, and adding this mixture fractionately.
2. Simultaneous addition of both coagulants.
3. Adding the iron salt previous to the alum, keeping up the relations used for the previous tests, i.e. $\frac{2}{3}$ of alum salt and $\frac{1}{3}$ of iron salt.
4. Addition of the iron salt in the proportion of $\frac{2}{3}$ of the total dose, and subsequently the alum until completing the dose.
5. Fractionation of the dose in three and four parts, with a one-minute interval between each fraction.

All these latter tests gave disappointing results as compared with those of the fractionated coagulation.

Fractionated coagulation furnishes the following advantages: (1) better quality of the settled water; (2) use of iron salts; (3) a more rapid decantation; and (4) saving of coagulant.

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The Water Supply System of San Francisco

By Nelson A. Eckart

SAN FRANCISCO, with a population of about 750,000 and water consumption of about 60 million gallons daily, derives its water supply from several different sources. For convenience these sources and the pertinent works and structures are frequently designated as the Hetch Hetchy Project, the Alameda System, and the Peninsula System. Located on the tip of a peninsula this city has from its founding been faced almost continuously with the problem of obtaining and maintaining an adequate supply of water to meet its continually increasing demands. As a small settlement, its initial requirements were supplied by local springs. As it grew into a city and developed, it became necessary to look to other sources and in 1851 a company was organized to transport water in tanks on barges from Marin County. In 1857, the San Francisco Water Works was organized to develop a supply from Lobos Creek, a small stream located within the present city limits. About 2 m.g.d. from this source were conveyed by flume and tunnel to Black Point on the northern shore of the city, from which point it was pumped to the Lombard and Francisco Street reservoirs for distribution. Although the Lobos Creek development does not now form a part of the municipal system, it is still utilized by the War Department as a source of supply for the Presidio; the Lombard and Francisco reservoirs constructed at that time, however, are still in service as a part of San Francisco's distributing system.

Small amounts of water were developed elsewhere locally, but realizing the limitations of these purely local sources the Spring Valley Water Works, which had been organized in 1860, pressed south into San Mateo County and began the development of what is now re-

A paper presented on October 27, 1939, at the California Section Meeting at San Francisco by Nelson A. Eckart, General Manager and Chief Engineer, Water Department, San Francisco.

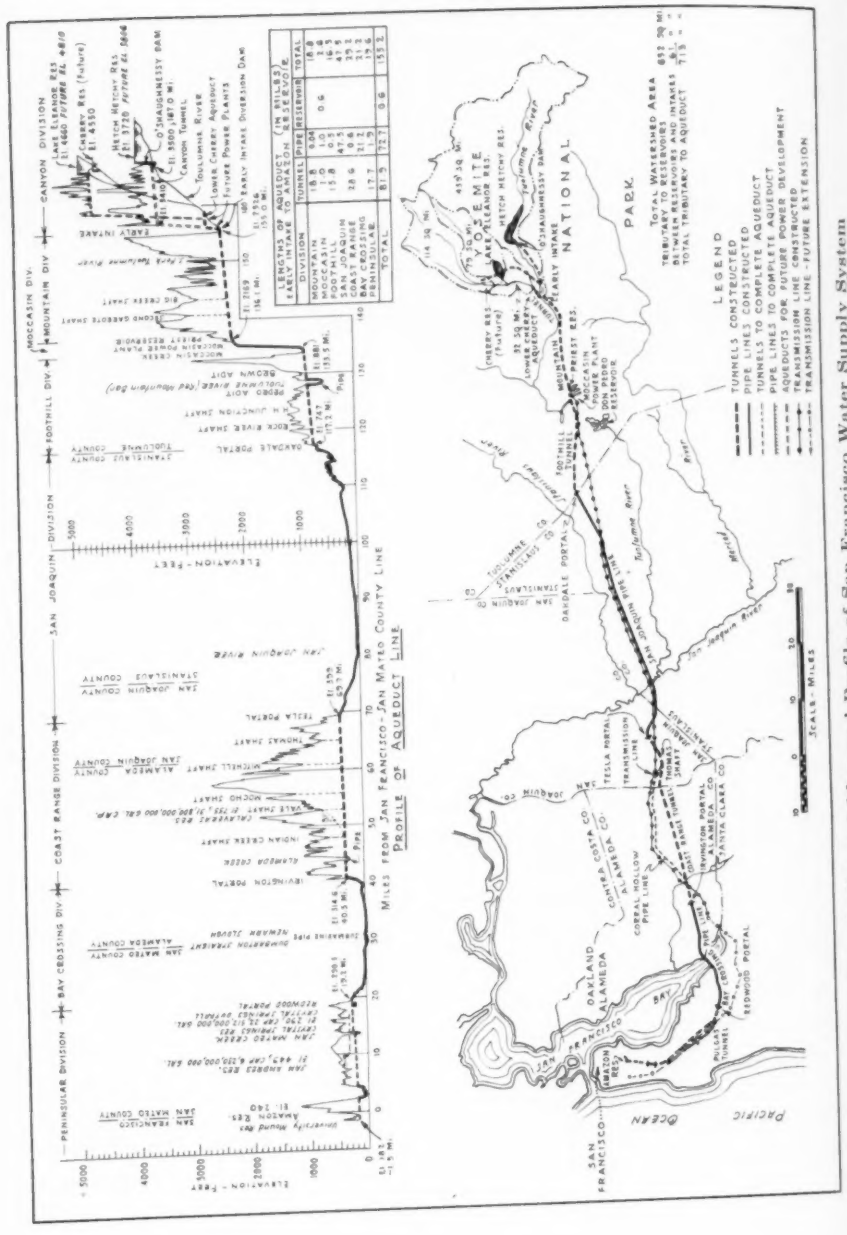
LENGTHS OF AQUEDUCT (IN MILES)
BY DIVISION
DIVISION | TUNNEL | PIPE | RESERVOIR | TOTAL
MOUNTAIN | 18.8 | 0.04 | | 18.8
MOCCASIN | 1.0 | 1.0 | 0.6 | 2.6
SOUTHFILL | 15.8 | 47.5 | | 47.5
SAN JOAQUIN | | 35.3 | | 35.3
COAST RANGE | 28.6 | 0.6 | | 29.2
BAY CROSSING | | 21.5 | | 21.5
PENINSULA | 17.7 | 2.7 | 0.6 | 19.6
TOTAL | 81.5 | 72.7 | 0.6 | 155.2
PROFILE OF AQUEDUCT LINE
MILES FROM SAN FRANCISCO-SAN MATEO COUNTY LINE
ELEVATION - FEET
PENINSULAR DIVISION
BAY CROSSING DIV.
COAST RANGE DIVISION
SAN JOAQUIN DIVISION
FOOTHILL DIV.
MOUNTAIN DIV.
MOCCASIN DIV.
CANYON DIVISION
LEGEND
TUNNELS CONSTRUCTED
PIPE LINES CONSTRUCTED
TUNNELS TO COMPLETE AQUEDUCT
PIPE LINES TO COMPLETE AQUEDUCT
AQUEDUCTS FOR FUTURE POWER DEVELOPMENT
TRANSMISSION LINE CONSTRUCTED
TRANSMISSION LINE - FUTURE EXTENSION
TOTAL WATERSHED AREA 652 SQ MI
TRIBUTARY TO RESERVOIRS 81 " "
BETWEEN RESERVOIRS AND INTAKES 119 " "
TOTAL TRIBUTARY TO AQUEDUCT 719 " "
YOSEMITE NATIONAL PARK
SAN FRANCISCO
OAKLAND
ALAMEDA
BAY
PACIFIC OCEAN
Scale - Miles

Fig. 1. General Map and Profile of San Francisco Water Supply System

ferred to as the Peninsula system. The first complete unit involved the construction of a reservoir at Lake Pilarcitos, from which by means of flume and pipe line gravity delivery of water was made into Lake Honda reservoir within the city. First delivery of water to San Francisco from Pilarcitos was made in July, 1862. Shortly after this, the company acquired certain water rights at Lake Merced, within the city limits, and continued its development of the Peninsula system by adding the San Andres reservoir in 1868, followed by the Upper Crystal Springs reservoir in 1877 and the Lower Crystal Springs dam in 1887. In the meantime, realizing the limitations of the Peninsula system, the Company had acquired extensive water rights together with the site of the present Calaveras reservoir in Alameda County. In 1887 the first pipe lines were constructed for transporting water from the Alameda Creek watershed from a point near Niles across the bay to a connection with the Peninsula system. Further developments in the Alameda system included the sinking of a number of wells in the Pleasanton area, the construction of the Sunol dam, with a system of underground filter galleries at this point. The Calaveras dam was constructed in 1921–25, and the Upper Alameda Diversion dam and tunnel completed by the city in 1932.

Throughout almost the entire existence of the Spring Valley Company, which served the city from 1860 until 1930, controversies had been more or less continually arising between the company and the city officials, generally as to rates and adequacy of service for fire protection and to the outlying districts. Numerous investigations and reports were made by engineers and committees on behalf of the city, looking toward the acquisition of an independent municipally-owned system and source of supply. These investigations finally led to the determination on the part of the city to acquire the necessary rights in the Tuolumne River and its tributaries, and the development of the Hetch Hetchy project. With the ultimate development of this system there will be sufficient capacity to meet the future requirements of San Francisco and the entire Peninsula area as far south as San Jose.

Construction of the Hetch Hetchy project was started by the municipality shortly after passage of the Raker Act in 1913, granting necessary permission for use of government land in connection with the project. First delivery of water from Hetch Hetchy to San Francisco's peninsula storage reservoir took place in October, 1934.

FIG. 1. General Map and Profile of San Francisco Water Supply System

In the meantime the city had faced the problem of constructing or acquiring a local water distribution system to distribute the Hetch Hetchy supply. This was solved by acquiring the system which had been operated since 1860 by the Spring Valley Water Company. Proposals for the purchase of the system were submitted to the voters on a number of occasions but these were defeated, until on May 1, 1928, a bond issue was voted by more than the necessary two-thirds majority. Due to the conditions of the bond market, some difficulty in arranging for the immediate sale of the bonds was experienced, but the purchase was consummated and the city acquired ownership of the Spring Valley system on March 3, 1930.

The newly-created San Francisco Water Department was at first administered under the Municipal Board of Public Works. Under a new charter which became effective January 8, 1932, jurisdiction was transferred to the San Francisco Public Utilities Commission, together with all other municipally-owned utilities including the Hetch Hetchy Water Supply, the Hetch Hetchy Power System, the San Francisco Municipal Railway, the San Francisco Airport and the lighting of streets and public buildings. The commission is composed of five members appointed for four-year terms by the mayor. Executive powers of the commission are vested in a manager of utilities appointed by the commission and subject to its pleasure. The San Francisco Water Department is organized as a department under the commission and is headed by a general manager and chief engineer named by the manager of utilities with the approval of the commission.

Water from the Hetch Hetchy project is delivered by a series of tunnels and pipe lines into the Crystal Springs reservoir where it augments the water produced from the Peninsula watersheds; likewise, the waters from Calaveras reservoir and the Alameda system are transported across the bay in the Hetch Hetchy aqueduct and delivered into Crystal Springs reservoir from which point a large portion of the City's requirements is delivered by gravity into University Mound reservoir for distribution in San Francisco.

The runoff from the San Andres and Pilarcitos watershed, supplemented by water pumped from Crystal Springs reservoir into San Andres reservoir, is delivered by gravity to the College Hill, Merced Manor, Lake Honda and Sunset reservoirs in San Francisco for distribution to the higher levels of San Francisco, it being necessary to pump a certain amount of water from these receiving reservoirs

in San Francisco to various tanks and reservoirs located throughout the city to serve the various districts.

The Peninsula System

The sources of supply of the Peninsula System consist of four reservoirs located in San Mateo County, with which are included those auxiliary sources, Lake Merced and the Sunset Wells, located in San Francisco. The four reservoirs, Pilarcitos, San Andres, Upper and Lower Crystal Springs, have a combined storage capacity of 29.5 bil.gal. and a productive capacity of approximately 16.0 m.g.d. The total combined watershed area tributary to these reservoirs is approximately 32 sq.mi., which for the most part is covered with a heavy growth of trees and brush. The average rainfall on the several catchment basins ranges from 29 in. at Crystal Springs to 49 in. at Pilarcitos.

The entire Peninsula watershed, with minor exceptions, is city-owned. It is fenced and patrolled and except for the resident keepers and patrolmen is uninhabited. Shortly after San Francisco acquired these properties the area was established as a state fish and game refuge, all of which makes for ideal conditions for the production and maintenance of a water supply of high sanitary standard. A brief detailed description of the several reservoirs and auxiliary sources is given below.

Pilarcitos Reservoir

The Pilarcitos reservoir, located on the western slope of the mountains midway between San Francisco Bay and the Pacific Ocean, 11 mi. south of the County line, has a capacity of 1.0 bil.gal. with water at flood line elevation of 695.

The watershed area naturally tributary to this storage basin is 3.8 sq.mi., which by diversions from adjoining areas and collecting flumes has been augmented by some 1.4 sq.mi. Being on the western slope and with elevations reaching 1,875 ft. this drainage area has the highest average annual rainfall, 49 in., and the most prolific production of the Peninsula reservoirs.

The dam is of the rolled-fill earth type with puddled clay core. It is 95 ft. high with crest length of 520 ft. at elevation 700. The upstream slope is $2\frac{1}{2}$ to 1 and downstream the slope is $2\frac{1}{4}$ to 1. The dam was originally completed to a height of 70 ft. in 1866, but later in 1874 was raised to its present height.

A new concrete spillway of overfall type, with a capacity of 1,200 sec.-ft., was completed recently. The three old manually-operated waste gates, which formerly served to control the overflow from this reservoir, have been maintained and are still available for use.

The outlet from the reservoir is through a brick and stone gate tower connecting with a small brick-lined tunnel at elevation 651.4 which pierces the ridge to the east of the basin. Originally, water from this reservoir was delivered by gravity through flumes and pipe directly to Lake Honda distributing reservoir in San Francisco. Later, the flume sections were replaced by a continuous pipe line, but in the earthquake of 1906 this pipe line, 36 in. in diameter, of riveted wrought-iron plates, which closely paralleled the San Andres fault line, crossing and recrossing it several times, was almost entirely destroyed and was never replaced. Under present operations water released from the reservoir is conveyed by tunnel and pipe to the San Andres reservoir. The runoff into this reservoir (Pilarcitos) is generally largely in excess of its storage capacity, so that as opportunity presents stored waters are released to San Andres to permit additional catchment. Water passing the spillway is picked up at the old Stone Dam about 2 mi. below and, with the drainage from this lower area, diverted by flume and tunnel to San Andres or to the Crystal Springs drainage as governed by the capacity of the connecting aqueduct.

San Andres Reservoir

The San Andres reservoir has a capacity of 6.0 bil.gal. with water elevation at 446.0 ft. It is located on a branch of San Mateo Creek about $2\frac{1}{2}$ mi. northeast of Pilarcitos.

There are approximately 4.4 sq.mi. of watershed area on this creek above the dam but this is supplemented to the extent of 9.2 sq.mi. by the diversion of drainage from Pilarcitos Reservoir and Creek previously referred to, and from another branch of San Mateo Creek, this latter by means of 3 small diversion dams and tunnels through the dividing ridge. In addition to the runoff from this 13.6 sq.mi. of drainage area, the supply of the San Andres reservoir is augmented as needed to meet the draft, by pumping from Crystal Springs reservoir. The water is delivered to the reservoir through what is known as the Crystal Springs-San Andres aqueduct, a concrete-lined canal about $6\frac{1}{4}$ mi. in length with a capacity of approximately 60 m.g.d. Water is delivered into the canal by the Crystal

Springs pump station, located below Crystal Springs dam, pumping through several parallel force mains.

The San Andres dam is similar in type and construction to the Pilarcitos dam. It was completed to its initial stage in 1870 and was increased in height in 1875 and later again in 1928. At present it is 105 ft. high with a crest length of 950 ft. at elevation 456.0. The upstream slopes are $3\frac{1}{2}$ to 1 and 2 to 1 and the downstream slopes 3 to 1 and $1\frac{1}{2}$ to 1. The upstream slopes are riprapped and protected from wave action by a heavy log boom.

Of interest in connection with this dam is the very satisfactory manner in which it withstood the 1906 earthquake. The San Andres fault line passed directly through the easterly abutment, but though there was a shearing movement of 8 ft. along the rift at this point, no damage to the embankment was sustained and the safety of the structure was unimpaired.

The dam when last raised was provided with a concrete lined spillway of the open channel type with a capacity of 2,500 sec.-ft. in addition to the old waste gate and tunnel structure which is still available with a capacity of 1,000 sec.-ft.

There are two outlets to the San Andres reservoir. The south, or Bald Hill outlet as it is generally known, as originally constructed was a brick lined gate tower or shaft. This was later replaced by a concrete structure through which the water released passes into the outlet tunnel at elevation 376.5. This tunnel which passes through the ridge to the east of the reservoir is brick lined, of approximately 5 ft. by 3 ft. elliptical section, 2,800 ft. long. The old brick gate structure which was immediately adjacent to the rift line was badly squeezed and cracked at the time of the 1906 earthquake, whereas the concrete structure located close by was not damaged. The tunnel leading from the gate structure was damaged in the immediate vicinity of the rift but was repaired and has been maintained in service ever since.

The north outlet was built in 1928 to provide increased discharge capacity and delivery to San Francisco via a new 54-inch transmission main laid at that time. In constructing this outlet care was taken to locate the entire gate structure east of the fault line, providing the necessary channel to the outlet screens by means of a wide open cut with flat slopes. The gate tower is a reinforced concrete structure with intakes at 401.5 and 425.0 ft. elevation. From the steel standpipe within the shaft, connection is made to the 54-inch steel outlet

pipe which is laid in a horseshoe-shaped concrete-lined tunnel 700 ft. long passing through the ridge to the basin.

Crystal Springs Reservoir

Crystal Springs reservoir with capacity of 22.5 bil.gal. at flow line elevation 288.0 is by far the most important of these Peninsula reservoirs. It was formed by the construction of a massive concrete dam across San Mateo Creek below the junction of the main branches. This dam is about 13 mi. south of San Francisco, the Skyline Boulevard being carried over its crest. The combined watershed area tributary to the Crystal Springs reservoir is 22.5 sq.mi. and with an average rainfall of 29 in. produces a yield of approximately 9 m.g.d. The reservoir is divided in two parts, Upper and Lower Crystal Springs, by an earth dam designated as the Upper Crystal Springs dam located about 3 mi. from the lower end of the lake. This dam, an earth-filled puddle clay core similar in type and dimensions to the Pilarcitos dam, was constructed in 1877 as the original development for the reservoir. With the completion of the main concrete dam, its usefulness passed except possibly to support a roadway across the lake.

The Crystal Springs dam, or as it is sometimes called Lower Crystal Springs dam to distinguish it from the earlier one, was constructed in the years 1887–88. It was raised a few feet in 1891 and again in 1911. It is of full gravity type, arched in plan, constructed of large interlocking blocks of concrete formed and poured in place; these blocks are of the general dimensions of 40 ft. long, 30 ft. wide and 8 ft. high. They are of irregular shape designed to afford an elaborate set of keyways and offsets. Alternate blocks were built and the space between later filled with concrete to minimize the effect of the concrete shrinkage. The dam as constructed is 154 ft. high with a base thickness of 176 ft., the section of the dam being designed to permit increasing its height some 45 ft. by building on the crest. The crest length is 600 ft. at elevation 288.85 with crest width of 40 ft.

The spillway is through a depressed section of the dam crest with concrete lip at elevation 280. Provision is made for flashboards to carry the lake surface 8 ft. above the lip. The spillway discharge is into a pool constructed at the toe of the dam below. The dam is an excellent example of good concrete construction. The concrete was mixed with a minimum water content and placed and tamped under rigid supervision, with proper attention to curing. Today, after 50

years' service, it appears to be as good as the day it was built. The splendid manner in which this structure, located very close to the fault line, weathered the 1906 earthquake without the slightest damage, is an excellent testimony for its design and construction.

Besides providing a catchment basin for the local run-off, Crystal Springs reservoir provides a terminal reservoir for the Hetch Hetchy aqueduct, and under present operations all water destined for San

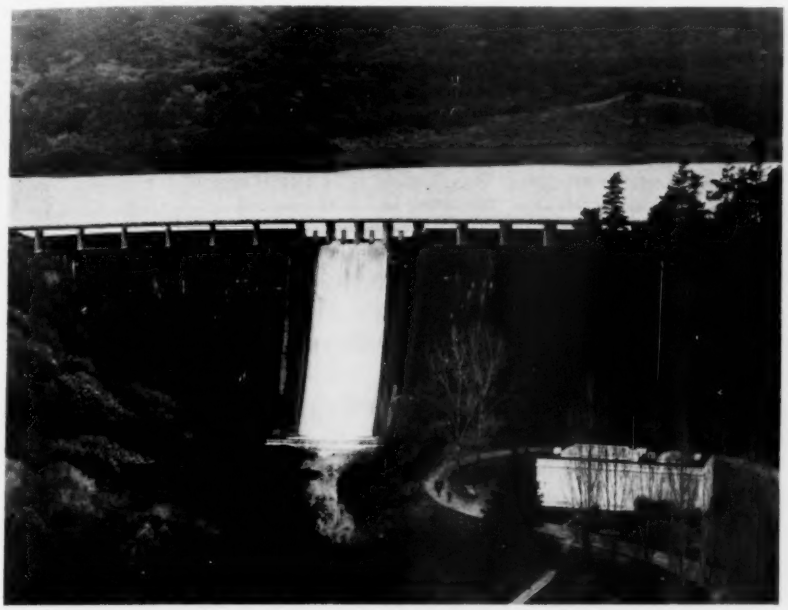

FIG. 2. Crystal Springs Dam: Arched gravity type with interlocking concrete blocks, constructed in 1888, 154 ft. in height, impounds 22.5 bil. gal. Serves as terminal reservoir for Hetch Hetchy aqueduct and supplies University Mound distributing reservoir in San Francisco by gravity.

Francisco, from both the Alameda system and the Hetch Hetchy system, is delivered into this reservoir basin.

There are two outlet towers or shafts controlling the release from Crystal Springs reservoir, both being located near the concrete dam. These towers, similar in design, are circular in shape, housing a steel standpipe with manually-operated gate valves controlling the inlets at the various levels. The original tower, completed in 1891, is of brick, 14 ft. in diameter with intakes at 3 levels, 177.9, 216.9 and 242.4 ft., respectively. From the bottom of the shaft the

connecting riveted-steel outlet pipe, 54 in. in diameter, leads through a brick-lined tunnel 360 ft. long. The second tower with its 3 reinforced concrete intakes was constructed in 1931, taking advantage of an extreme low water condition in the reservoir to perform the work. The outlet tunnel was not constructed, nor the standpipe installed, until 1937. This shaft is 16 ft. in diameter with intakes likewise at 3 elevations, 184.0, 226.0 and 255.0 ft. The tunnel through which the 78-inch outlet pipe is laid is 650 ft. long and lined with reinforced concrete.

Peninsula Transmission Conduits

There are two groups of transmission mains used to convey water to San Francisco from the Peninsula reservoirs, one group leading from San Andres to serve the higher level districts in San Francisco, the other from Crystal Springs to serve the large downtown area and the low-level districts. The mains from Crystal Springs reservoir consist primarily of two pipe lines leading from connections with the reservoir outlets to the University Mound distributing reservoir located in the southeast corner of San Francisco with elevation of water surface at 172.0 ft. The original line consists of some 17 mi. of 44-inch diameter wrought-iron riveted pipe with plates of No. 6 and No. 7 gage (about $\frac{3}{16}$ and $\frac{11}{64}$ in.). The route of this line follows down San Mateo Creek canyon, passing through the town of Hillsborough, then follows along the El Camino Real past the Millbrae pump station, and crossing the marsh lands near South San Francisco, skirts the westerly shore of San Francisco Bay to the terminal reservoir. Approximately $1\frac{3}{10}$ mi. of this line is supported on timber trestles crossing the marshes, and about $\frac{1}{2}$ mi. of it is through brick-lined tunnels.

The new 60-inch diameter line was constructed in 1937. While it follows generally the same route as the original line, opportunity was taken where possible to avoid some of the bad marshy ground near South San Francisco where experience has indicated damage might be sustained in case of earthquake. This line is of welded steel with $\frac{3}{8}$-inch plate thickness. In three locations aggregating 4,000 ft., where the line pierces the hills the pipe is laid in a tunnel, the space between the pipe and the tunnel walls being filled with concrete and grouted. About $\frac{1}{2}$ mi. of this pipe is supported on pile trestles. Between Burlingame and Millbrae about 3 mi. of an existing 54-inch pipe line constructed in 1903 as a portion of the Alameda pipe line, is incorporated as a part of the system.

Just below Crystal Springs dam the two pipe lines are connected through a ring manifold, from which the Crystal Springs pump station can pump from either or both lines to San Andres and water can be supplied to either transmission main from either or both outlet towers. The two lines are inter-connected through control valves at Burlingame and Millbrae, with additional cross-connections being made at Sierra Point. This new 60-inch Crystal Springs line is also cross-connected with the 54-inch main from San Andres reservoir at Baden. The two lines have a combined maximum capacity, for delivery to University Mound reservoir by gravity, of 74 m.g.d., 21 m.g.d. from the 44-inch, and 53 m.g.d. from the 60-inch line.

From the outlet of the Bald Hill tunnel of the south tower of San Andres reservoir a 44-inch riveted wrought-iron pipe leads to the Millbrae pump station, and extends along the El Camino Real to Baden, a total distance of about 5.1 mi., where it branches into two 30-inch lines. One of these lines extends a distance of 8.1 mi. to the College Hill reservoir at elevation 255.0 in San Francisco, the other leads to the new Merced Manor reservoir, a distance of 5.2 mi., where it discharges at elevation 187.5. The original line to Baden and the College Hill branch were constructed in 1870 as 30-inch lines. The portion of the line from the outlet to Baden, due to corrosive soil conditions, had to be replaced between 1895 and 1898, at which time a 44-inch line was installed between those points. The 30-inch line to College Hill is still in service but is giving trouble due to leakage in certain areas. The Baden Merced branch was constructed in 1907; it has a capacity of 14 m.g.d. The College Hill branch, while theoretically capable of delivering 15 m.g.d., now, with due respect to its age, is operated at a reduced capacity of 7.5 m.g.d.

From the North San Andres outlet a 54-inch diameter lock-bar steel pipe leads directly to Baden, from which point it parallels the Baden Merced branch for some distance and then extends northerly up Junipero Serra Boulevard and Portola Drive to Lake Honda. This line which is 10.9 mi. in length has plate thicknesses varying from $\frac{1}{4}$ in. to $\frac{1}{2}$ in. A 54-inch welded steel pipe 8,800 ft. long, connecting with this line at the junction of Junipero Serra and Sloat Boulevard was constructed in 1935 to supply the new Sunset reservoir at elevation 385.5. The two San Andres transmission mains are inter-connected at Baden, at which point present plans contemplate the installation of a pumping station which will provide for boosting water from the Crystal Springs system to the San Andres system.

At Millbrae, a pump station with 9 m.g.d. steam- and 5 m.g.d. electric-driven pump capacity provides for boosting water from the Crystal Springs system to the San Andres system. Until a few years ago operation of these pumps was more or less continuous to supplement the gravity delivery from San Andres reservoir to San Francisco, but with the construction of the Crystal Springs-San Andres canal, replacing the earlier wooden flume with increased capacity, normal pumping operations were transferred to the Crystal Springs pump station, and Millbrae now serves as an emergency unit only. Within San Francisco, a line of 44-inch and 36-inch pipe recently laid, and referred to as the cross-town line, provides for delivery of water from the 54-inch San Andres line to College Hill reservoir and also to the University Mound district which is normally supplied through the University Mound reservoir from Crystal Springs.

With water in the San Andres reservoir at the spillway level, the 54-inch line will deliver by gravity 32 m.g.d. to Lake Honda reservoir, or 37 m.g.d. to the new Sunset reservoir. This, with the deliveries to College Hill and Merced Manor reservoirs, provides for gravity delivery from San Andres reservoir at present of 58.5 m.g.d. With the available capacity of 74 m.g.d. from Crystal Springs, the total available capacity from the Peninsula reservoirs to San Francisco is 132.5 m.g.d.

San Francisco Sources

Lake Merced, located in the westerly portion of San Francisco just north of the county line, has a storage capacity of 2.5 bil.gal. The lake which is slightly above sea-level, derives its supply from springs which are fed by the rainfall on the drainage area percolating through the sandy soil which forms the catchment basin. The lake has a normal productivity of 3.5 m.g.d. For many years it contributed normally to San Francisco's supply. Surface drainage is diverted around the lake by means of a system of flumes and a brick-lined canal discharging through a tunnel to the ocean. Water from this lake may be delivered into the Lake Honda system through what is called the City Pumps station where two steam pumps, each of 7.5 m.g.d. capacity, are maintained and may serve to pump either Lake Merced water or San Andres water to Lake Honda. As the quality of water from this lake is somewhat inferior it is not normally utilized, and is now maintained solely as an emergency supply. Recently,

it has been stocked with bass and has been opened for fishing and boating under restricted conditions.

Another auxiliary source of supply located in San Francisco is the Sunset Wells system. During the critical dry season starting in 1930, some 20 wells were sunk in the Sunset district about $\frac{1}{2}$ mi. from the ocean. These wells have an average depth of 250 ft. through sand and gravel, and although they were pumped con-

FIG. 3. Calaveras Dam: Massive earth and rockfill, completed in 1925, height 220 ft., impounds 31.5 billion gallons. Released water, after passing through aerator, is conveyed to Crystal Springs reservoir through Hetch Hetchy aqueduct. Note inclined arches of riprap.

tinuously for several years at a rate of about 6 m.g.d. the water table was maintained at a level slightly below sea-level, without any noticeable salt intrusion. The water, which was of excellent quality, was pumped from the wells into a reinforced concrete receiving tank of 500,000 gal. capacity from which it was pumped through electric-driven pumps into the Lake Honda System. Use of the station was discontinued after the period of emergency had passed and with the wells is now maintained only as an auxiliary source of supply.

The Alameda System

The Alameda sources all lie within the drainage area of Alameda Creek, located in the Coast Range mountains on the east side of San Francisco Bay. The principal source of supply in this area is the Calaveras Reservoir which is supplemented by two underground sources, one in the Sunol Valley, the other in the Livermore Valley.

FIG. 4. Upper Alameda Creek Diversion Dam: A reinforced concrete, hollow slab and buttress type structure diverts the runoff from 35 sq. mi. of watershed through 2 mi. of tunnel into Calaveras Reservoir. Constructed by the Department in 1932.

The Calaveras reservoir impounds the runoff of 100 sq.mi. of watershed of Calaveras Creek and the Arroyo Honda, originating with Smith and Isabel Creeks on the slopes of Mt. Hamilton extending up to elevation 4,209 ft. The rainfall ranges from approximately 23 in. at the dam to over 30 in. on Mt. Hamilton. In addition to the Calaveras watershed, the drainage from approximately 35 sq.mi. of Upper Alameda Creek is diverted through a tunnel into the Calaveras storage. This reservoir with flow line at elevation 753 impounds 31.5 bil.gal. The yield including the Upper Alameda

Creek Diversion, taking into consideration the recent critical period, is estimated at 30 m.g.d.

The dam which is located about 9 mi. south of the town of Sunol is of the earth and rock-fill type. The lower portion was constructed by the hydraulic-fill method, and the upper part with a rolled-clay core supported on either side by loosely dumped material containing a large proportion of rock. The height of the dam is 220 ft. above bed rock; the crest of the dam at elevation 775 is 1,200 ft. in length

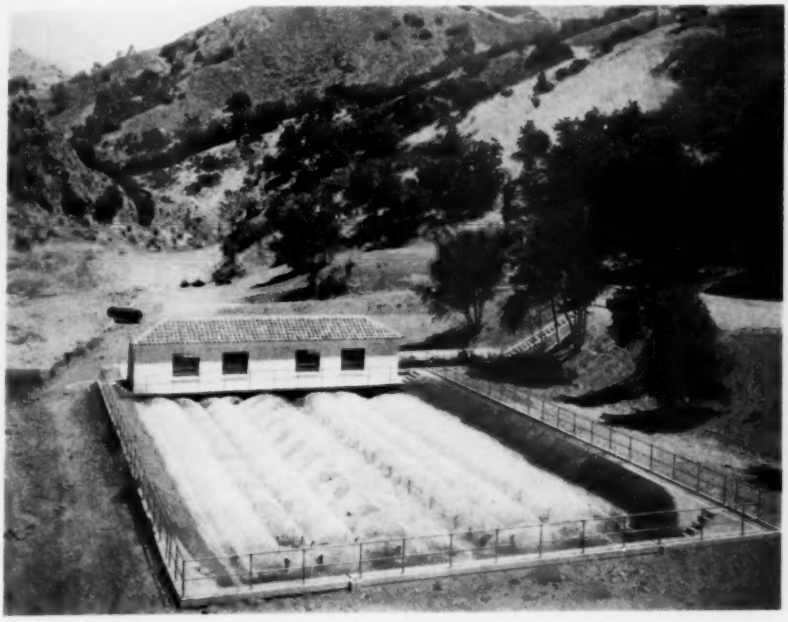

FIG. 5. Calaveras Aerator: Capacity 80 m.g.d., located immediately below dam, aerates water released from Calaveras before entering Hetch Hetchy aqueduct.

with a width of 75 ft. The upstream slopes are 5 to 1 and 3 to 1 with berms at 690 ft. elevation and at 738 ft. The downstream slope averages approximately $2\frac{3}{4}$ to 1 with two wide berms. The thickness of the dam at the base is 1,500 ft. The volume of material is approximately 3,500,000 cu.yd. The upstream face of the dam is riprapped with hand-placed rock, the upper portion for aesthetic reasons being laid as a series of inclined arches.

The spillway with its lip at elevation 753 is designed to pass a flood of 22,000 sec.-ft.

The outlet tower, located at the west abutment of the dam is of reinforced concrete with steel standpipe and control valves located within the tower. Intakes are provided at three different levels, the lowest one being at elevation 605.5. From the bottom of the outlet tower a concrete-lined tunnel extends through the abutment a distance of 1,350 ft. in which a 48-inch outlet pipe is laid.

Just below the tunnel the outlet pipe branches, one branch leading downstream to a 22-inch needle valve, through which water may be released into the stream channel to flow down to the Sunol filter galleries and the other branch leads to an aerating basin, with a capacity of 80 m.g.d. from which it flows in pipe to a connection with the Alameda Creek siphon crossing of the Hetch Hetchy aqueduct to flow by gravity across the bay to Crystal Springs reservoir.

Sunol Filter Galleries

At the upper entrance to Niles canyon lies a gravel-filled depression of about 1,300 acres known as the Sunol Valley through which the Alameda Creek drainage flows. Below this area at the entrance to the canyon, a concrete restraining barrier about 28 ft. high called the Sunol dam has been built which serves to back up the creek flow and keep the gravels flooded. A system of underground galleries and perforated concrete pipe provides for collecting the water percolating through the gravels. The normal dependable yield from this source is approximately 5 m.g.d., under flood conditions, however, these galleries will produce clear water up to 20 m.g.d.

In the Livermore Valley in the vicinity of Pleasanton an extensive area of deep gravel provides an underground gathering basin, from which the city has the right to withdraw and divert 15 m.g.d. Some 100 wells have been sunk in these gravels ranging in depth generally from 200 to 400 ft., one, however, being carried to 734 ft. During the critical period from 1930–34 about 20 of these wells were operated to produce the allowable draft from a constantly lowering water table. At present only about 4 wells are operated to supply the town of Sunol and a few local consumers. As a result, the water table has been restored from its low point of 150 ft. to practically its original level. The dependable continuous yield of this field is estimated at 6 m.g.d., but due to the extent of the underground storage the maximum permissible draft can be delivered over an extended cycle of dry years. Waters gathered from the Pleasanton well field are delivered to Sunol through 28,000 ft. of 30-inch pipe where they join

with the waters from the filter galleries in the Sunol Water Temple, a structure of classic design erected over the open gathering basin and weirs.

Alameda Transmission System

Waters leaving the Sunol Temple enter the Sunol aqueduct through which they are delivered to the Niles reservoir, about 5 mi. distant, at elevation 181.0. This aqueduct, which has a capacity of 70 m.g.d., consists of a series of short tunnels and reinforced-concrete covered flumes, the length being about equally divided between the two types of structures. The Niles reservoir, formed by excavation and embankment, is concrete lined and covered with a wooden roof. It has a capacity of 5 mil.gal.

Two pipe lines lead from Niles reservoir. One, consisting of 15,600 ft. of 44-inch diameter steel pipe, was installed by the Spring Valley Company in 1924 to provide a connection with the 60-inch Bay Crossing pipe of the Hetch Hetchy Project, which unit of that project was constructed by the city at that time under agreement with the company to provide increased capacity for transporting water across the Bay to San Francisco. The other is the original, so-called Alameda pipe line, which served to convey Sunol water across the bay to a connection with the Crystal Springs pipe line at Millbrae. The total length of line to Millbrae was 31.3 mi. of which 26.87 mi. consisted of 36-inch riveted wrought-iron pipe, 3.17 mi. of 54-inch wrought-iron riveted, and 1.29 mi. of submarine flexible-joint crossing Newark Slough and San Francisco Bay. The pipe in these crossings consisted of two lines of 16-inch and two lines of 22-inch galvanized tubing with cast-iron ball and socket joints. Several long sections of the Alameda line, where laid in heavy soil, have been taken out of operation due to corrosion, but cross-connections with one of the Hetch Hetchy pipe lines, one near Newark and the other at Ravenswood, have enabled utilization of the Bay Crossing section of this old line for emergency purposes. The two 16-inch submarines were abandoned and in part removed to clear the way for the laying of the second and third submarine pipe lines of the Hetch Hetchy aqueduct. This recovered pipe after nearly fifty years' service under the waters of San Francisco Bay, was cleaned and dipped and relaid as a portion of the distribution system in San Francisco.

Water conveyed through the Alameda line formerly was boosted at the old Belmont pumps for delivery into the Crystal Springs pipe

at Millbrae. Under present operating conditions any water from the Sunol sources not used for consumption en route is pumped through one of the Hetch Hetchy pipes into Crystal Springs. The Belmont pumps have been abandoned, and water from Crystal Springs is now fed back into the Alameda line through a connection at Burlingame to Belmont. Present plans provide for rehabilitating and extending this line back from Belmont to a connection with the Hetch Hetchy line at the Redwood City gate house, which will enable

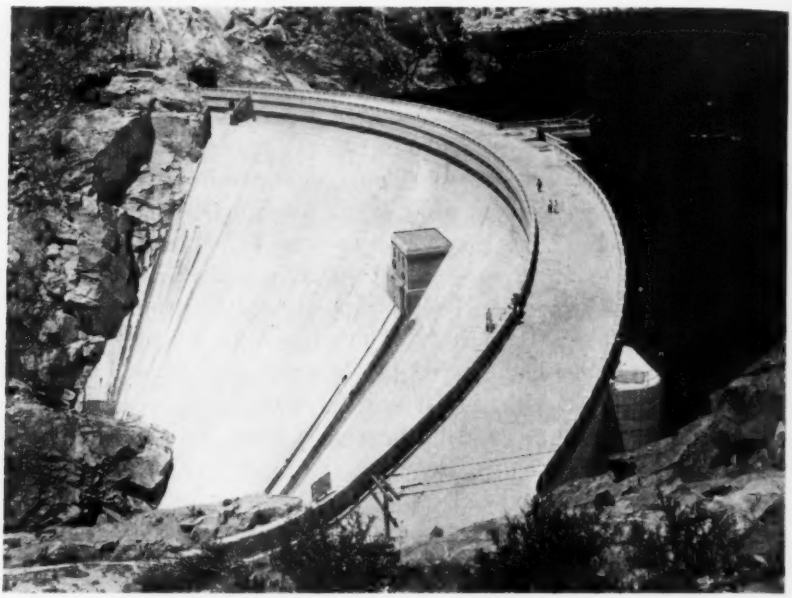

FIG. 6. O'Shaughnessy Dam: A concrete gravity arch structure, 430 ft. high above bed rock, impounds 117 bil. gal. Originally constructed in 1923, was raised 85.5 ft. in 1938.

Crystal Springs storage to be utilized to provide a more dependable supply for the area in the lower peninsula.

The Hetch Hetchy Project

The Hetch Hetchy project is designed ultimately to supply 400 m.g.d. for San Francisco and, in addition, develop some 250,000 h.p. of hydro-electric power. Hetch Hetchy reservoir, formed by the construction of O'Shaughnessy Dam, provides catchment for some 459 sq.mi. tributary to the Upper Tuolumne, and Lake Eleanor

receives the runoff from another 79 sq.mi. tributary to Eleanor Creek. Both of these sources are entirely within Yosemite National Park. Additional sources include some 114 sq.mi. of watershed which can be diverted through a flood canal from Cherry Creek and 32 sq.mi. below the main diversions from Eleanor and Cherry Creek, both of which can be conveyed into the main aqueduct.

The watershed is composed of rugged granite mountains, all above elevation 3,700, and 92 per cent of which is above elevation 6,000. Mt. Lyell, the highest peak, reaches an elevation of 13,000 ft. All of the land belongs either to the city or the United States Government and is practically uninhabited.

O'Shaughnessy Dam, a concrete gravity arch structure of radius 700 ft., was completed to a height of 344.5 ft. above bedrock in 1923, creating a storage capacity of 67 bil.gal. with the lake at spillway elevation of 3,720 ft. The foundation and the center block, containing the outlet pipes, were constructed so that the dam could be raised 85 ft. In 1938, the work of raising the dam this additional height was completed, increasing the storage capacity to 117 bil.gal. The completed dam has a crest 900 ft. long and 25 ft. wide and is 308 ft. thick at the base.

Although increase of the height of O'Shaughnessy Dam had been provided for in the design of foundations for the original construction completed in 1923, it was not at that time anticipated that the increase to the ultimate height would be desirable for many years. Eleven years of operation of the Moccasin Creek power plant with water stored by O'Shaughnessy Dam demonstrated, however, that in successive years of deficient rainfall there was not a sufficient carry-over of storage to permit a continuous operation of the power system at full capacity. Rainfall records for an eleven-year period indicated that enlargement of the storage capacity would permit continuous operation and that this would bring the city an additional net annual revenue averaging $225,000. This prospect of additional revenue together with the possibility of financing the construction with the assistance of federal funds through the Public Works Administration made it economically desirable to increase the dam to its ultimate height at a time far earlier than had been anticipated.

The task of economically adding this additional height to a structure of the type and magnitude of this dam was in some respects perhaps unprecedented. It involved special construction features to insure that upon completion, the new portion of the structure

would immediately take up and continue to bear its proper share of the load, and that the entire mass of the enlarged dam would act under stress in practically the same manner as if originally built to the final dimensions.

The general dimensions of the addition were fixed by the original design, and no substantial change was found desirable after thorough review. The downstream face of the original dam was finished in steps 5 ft. high to assist in bonding the new structure. Construction joints in the original dam were spaced at 90-foot centers, which

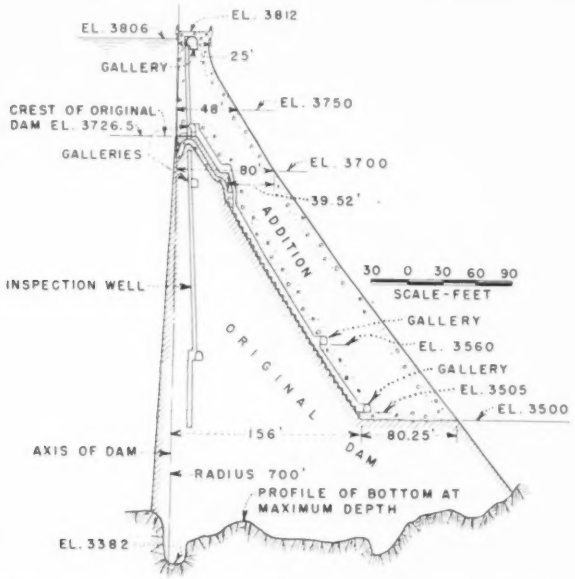

FIG. 7. O'Shaughnessy Dam: Typical section showing original work and recent addition.

was representative of the practice at the time. These joints were continued in the new structure, but additional intermediate joints were also provided, making the average spacing in the addition 45 ft. Thorough drainage is provided between the old and new concrete by means of a drain on each 5-foot step.

The most interesting and important design problem was to make the old and new concrete act as a homogeneous structure. Since the original concrete was over 13 years old when the new was poured, it presumably had undergone practically all the shrinkage that normally occurs in setting, as well as any other physical changes. Further-

more, due to the necessity of maintaining a head of water in the reservoir the addition had to be constructed while the old concrete was under stress. If the new concrete had merely been poured against the old structure, the older concrete, because of its superior characteristics due to age, might have become greatly overloaded. For these reasons it was necessary to bring the new concrete to approximately the same condition as the old, within the construction period, and to provide for satisfactory distribution of stress. This was accomplished by several expedients:

(1) A concrete mix was designed so that it would initially be of a somewhat higher grade than the old concrete when the latter was poured. This was accomplished by use of improved present-day methods without increasing the cement content (1 bbl. per yd.) over that of the old concrete.

(2) By artificially cooling the new concrete, in several months a temperature reduction was secured that would normally, in such a mass, require many years. The arrangement for cooling was substantially the same as that used at Hoover Dam. Connections were so arranged that cooling water could either be taken from the reservoir direct or through a refrigerating plant. Rate of cooling was controlled by adjusting the rate of flow in the cooling pipe, using observed temperature as a basis for change.

The bulk of the new concrete was not poured against the downstream face of the original structure but was supported by ribs running up the slope. These ribs were 2 ft. wide and spaced at intervals of about 22 ft. These 20-foot slots between the ribs were not filled until the main body of the new concrete had been permitted to shrink to a predetermined degree. Concrete poured in the slots later was also artificially cooled.

Construction joints in the new section of the dam were grouted through a system of pipes, and those in the old section through holes drilled in the concrete.

Contact surfaces between old and new concrete were thoroughly roughened to assist in securing proper bond. For further mechanical bond, $1\frac{1}{4}$ in. square anchor bars were set in $2\frac{1}{4}$-in. holes drilled in the downstream face of the old structure at intervals of 5 ft. vertically and averaging $2\frac{1}{2}$ ft. horizontally. These bars are 15 ft. long and project 10 ft. into the new structure.

The spillway for the enlarged dam, located near the south abutment is of the open-channel type with 20,000 sec.-ft. capacity. This

replaces the old siphon-type spillway of the original structure, the siphon outlets being sealed when the dam was raised.

Water can be released from the reservoir by means of 14 control gates and outlets. Six of these are 60 in. in diameter; six, 36 in.; and two, 72 in. All are of the manually controlled needle type, except one 72-inch valve which is of the butterfly type.

The enlargement of the O'Shaugnessy Dam was carried out as a P.W.A. project under the direction of L. T. McAfee, Chief Engineer of the Hetch Hetchy Water Supply, a department of the Public Utilities Commission.

Located about 5 mi. to the northwest of Hetch Hetchy, Eleanor dam was constructed on Eleanor Creek about a mile below the outlet of the original lake. The dam, a buttressed-arch structure 1,260 ft. long and 70 ft. high, with crest elevation at 4,661, raised the level of the lake 35 ft. and provides 9 bil.gal. storage. The central portion of the dam consists of 20 inclined arches with spans of 40 ft. The ends of the dam are composed of full gravity sections. Stored water is released through four 24-inch sluice gates.

The dam was constructed in 1918 to provide for continuous operation of a 2,400 kw. power plant at Early intake which provided electric energy for construction purposes on the project and which has since been continued in service for revenue.

Mountain Division Aqueduct System

Water released from Hetch Hetchy reservoir flows about 12 mi. down the Tuolumne River to the Early intake diversion dam at which point it is joined by the waters from Lake Eleanor and diverted into the mountain aqueduct tunnels. Lake Eleanor water is conveyed to Early intake via Eleanor Creek and Cherry Creek, a distance of 8 mi., to a small diversion dam which turns it into the Lower Cherry aqueduct. This aqueduct which has a capacity of 200 sec.-ft. and is $3\frac{1}{4}$ mi. long, consists of a series of tunnels, flumes and sections of open concrete-lined canal.

Early intake diversion dam, which diverts the flow into the Mountain Division tunnel, is a thin concrete arch structure 262 ft. long and 55 ft. high above stream bed, with crest at elevation 2,356. The intake for the tunnel is at elevation 2,326.

Entering the Mountain tunnel the water travels 19 mi. to Priest reservoir. This tunnel is of a concrete-lined horseshoe section 10 ft. 3 in. in maximum dimensions for a distance of approximately 11.6

mi. and for some 7.2 mi. is of unlined rectangular section 13 ft. 6 in. by 13 ft. 4 in. and the balance is of varying sizes. Minimum thickness of concrete lining is 6 in. The tunnel is driven on a uniform slope of 8 ft. per mi. and is designed to carry more than 400 m.g.d., the ultimate development of the Hetch Hetchy project.

Priest reservoir serves as a regulating reservoir for the Moccasin power house. Its function is to regulate the flow of water, which it receives at a constant rate of approximately 620 sec.-ft., and dis-

FIG. 8. Early Intake Diversion: A thin arch dam across the Tuolumne River serves to divert water released from O'Shaughnessy and Eleanor into the Mountain Division Tunnel of the Hetch Hetchy aqueduct.

charge it at a variable rate of from 0 to 1,240 sec.-ft., through the Moccasin power tunnel and penstocks, to the turbines, according to the demand for power. This reservoir, having a capacity of 760 mil.gal., was formed by constructing an earth and rock-fill dam, 1,160 ft. long and 145 ft. high above stream bed across Rattlesnake Creek. The crest is 20 ft. thick and at elevation 2,245. A jointed, flexible type concrete core wall extends 15 ft. into bedrock. The upstream slopes of $2\frac{1}{2}$ to 1 are covered with hand-placed riprap.

Moccasin Tunnel and Penstock

Leaving the Priest reservoir the water passes through one mile of tunnel and then drops 1,316 ft. through the penstocks to the Moccasin power house. The penstocks start from a reinforced concrete surge chamber 40 ft. in diameter and 160 ft. high. The floor elevation is 2,112. At the start there are two 104-inch pipes, but part way down the hill these branch to four 66-inch pipes and enter

FIG. 9. Moccasin Power Plant: Utilizes a drop of 1,316 ft. to operate four 20,000 K.V.A. waterwheel-driven generators. View shows surge chamber on ridge, the connecting penstocks, switch structure, power house and Moccasin re-regulating reservoir with Foothill Tunnel control gate in the foreground.

the power house as eight 36-inch pipes to serve the four 20,000 kw. water-wheel-driven generating units. These units deliver energy at 11,000 volts to the transformers which step the voltage up to 120,000 volts for transmission over a double-circuit steel tower line 98½ mi. to Newark where it is consigned to the Pacific Gas and Electric Company, the City's agent for distribution in San Francisco. Passing through the water wheels the water discharges into the Moccasin re-regulating reservoir which provides pondage for the

variable discharge from the power house and permits a steady flow to enter the Foothill tunnels, thus serving as a forebay for the closed aqueduct leading from this point to Crystal Springs reservoir. The reservoir has a capacity of 160 mil.gal. and is formed by an earth and rock-fill dam 855 ft. long, 50 ft. high above stream bed, with crest at elevation 927. Flow from the reservoir into the aqueduct is controlled by two 6x8½-foot electrically-operated slide gates in a reinforced concrete tower directly over the portal of the Foothill tunnel. To avoid pollution of the water supply, surface drainage tributary to Moccasin Creek is diverted by a concrete dam just above the reservoir and bypassed through a concrete conduit laid on the reservoir floor and extending below the Moccasin Dam.

Foothill Division Tunnels

Entering the Foothill tunnel the water flows 15.8 mi. west to the edge of the San Joaquin valley. The tunnel invert is at elevation 881 at the inlet and elevation 747 at the Oakdale portal outlet. This tunnel is on a grade of 8 ft. per mi. and like the mountain tunnels will carry in excess of 400 m.g.d. The tunnel sections are similar to the mountain tunnels, about one-half being the concrete lined horseshoe section and the remainder being of the rectangular unlined section with flat arched roof. About 5¼ mi. from the entrance portal, the tunnel emerges into the canyon of the Tuolumne River, which is crossed by the Red Mountain bar siphon. This siphon is a riveted-steel pipe ½ mi. long, 9 ft. 6 in. in diameter, and of a maximum plate thickness of ¾ in. The lower portion of this siphon, where it passes through the basin of the Modesto and Turlock Irrigation District's Don Pedro reservoir, is lined with 1½ in. of cement and encased in concrete 18 to 24 in. thick.

Future plans contemplate the construction of a power plant at this site to utilize the flow which may be diverted from Hetch Hetchy and not at the time required for water supply to San Francisco. In keeping with this plan a Venturi meter has been installed at this location, together with control gates which provide for regulating the flow in the tunnel and aqueduct below this point.

From Oakdale portal at elevation 747, the aqueduct crosses the San Joaquin valley to Tesla portal at elevation 399, a distance of 47½ mi. as a steel pipe line. The diameter of this pipe varies from 56 to 66 in. and plate thickness varies from $\frac{5}{16}$ to ½ in. All shop seams are electrically welded and field seams are riveted. The

capacity of this initial pipe installation is approximately 70 m.g.d. An ultimate capacity of 400 m.g.d. will be provided by the construction of additional lines, probably three 72-inch pipe lines. At the San Joaquin River, and at Elliott Cut, both of which are navigable streams, the pipe is depressed below stream bed, the bottom being 15 ft. below low water and resting on concrete capped timber piling. It is encased in a reinforced concrete jacket 6 in. thick and lined with $1\frac{1}{2}$-inch cement mortar. Automatic stop and relief valves are installed at Cashman Creek (mile $7\frac{1}{2}$) and at Elliott Cut (mile $25\frac{1}{2}$).

Coast Range Tunnels

Tesla Portal marks the entrance to the Coast Range tunnels. The first section is a continuous 25-mile tunnel emerging at Alameda Creek which is crossed by 0.6 mi. of steel siphon. The aqueduct then continues as another $3\frac{1}{2}$-mile tunnel bringing the water through the Coast Range Mountains to the Irvington Portal, elevation 316, near Mission San Jose in Alameda County.

The finished tunnel is 10 ft. 6 in. in diameter, of circular section, with concrete lining varying from 6 in. to 3 ft. in thickness depending upon the ground conditions. The grade is about 3 ft. per mi. and the capacity is 250 m.g.d. Ultimate development will require another similar parallel tunnel. The 25-mile tunnel was driven from five shafts and two portals and the $3\frac{1}{2}$-mile tunnel was worked from two headings. Shafts varied in depth from 300 ft. to 823 ft. These are offset about 95 ft. from the tunnel line, being located about midway between the center lines of the present and proposed future tunnels. Cross-cuts were driven to both tunnel lines, these are concrete lined for permanence, insuring availability for use in driving the second tunnel. During construction these provided space for charging the electric locomotives and for shop purposes.

Extremely heavy swelling ground was encountered in many locations throughout the tunnel and in some instances the swelling took place so rapidly that it became necessary to hold the ground preparatory to concrete lining by a technique developed on the job and involving the placing of annular sections of gunite concrete in lieu of timbers. In some instances a clear space of as much as one foot was left between the gunite rings and the tunnel walls as excavated, in order to permit the quick-setting cement to develop the necessary strength before the ground could squeeze in to press against the ring. The gunite used developed a strength of 6,000 lb. per sq.in.

in 28 days and in some instances tests went as high as 10,000 lb. Concrete normally was designed for 3,000 lb., occasionally for 4,000 lb. Methane gas encountered throughout the tunnel added to the difficulties in driving. Overflow shafts are provided at Tesla Portal and at each side of Alameda Creek to provide for air and pressure relief while regulating the flow.

The Alameda Creek siphon, separating the two sections of the Coast Range tunnels, is a 69-inch steel-cylinder reinforced-concrete pipe 0.6 mi. long. A connection is provided in this siphon section to permit the water from Calaveras reservoir to enter the Hetch Hetchy aqueduct by gravity. The conduit from Calaveras consists of 6 mi. of 44-inch welded steel pipe.

Bay Crossing Division Pipe

From the Irvington portal two parallel pipe lines, having a combined capacity of 114 m.g.d., convey the water across San Francisco Bay to the Pulgas tunnel, some 21 mi. west, through which tunnel it is discharged into the Crystal Springs reservoir. The first of these lines, completed in 1925, is of 60-inch riveted steel pipe except for a distance of 1.3 mi. immediately west of the portal which is 57-in. steel cylinder reinforced-concrete pipe laid in 1933. The second, completed in 1936, for a distance of approximately 5 mi. consists of 62-inch steel cylinder reinforced-concrete pipe, with the remaining portion, except for the actual crossing of the bay, of 66-inch welded steel. The navigable channel of San Francisco Bay is crossed by means of one 42-inch cast-iron flexible-jointed submarine pipe, laid in 1925, and two 54-inch welded steel pipes, laid in 1936. These are encased in concrete and supported on piling. West of the channel the pipes all enter a concrete caisson from the bay bottom in which they rise vertically to emerge through a manifold from which they are carried to the west shore on a steel truss bridge consisting of thirty-six 105-foot spans. Bridge piers and the caisson are of reinforced concrete supported on piling. Water-wheel-operated gates located in the caisson with cross-connections permit any combination of submarines to be connected to either pipe. On the steel truss bridge the original pipe line is 60-inch riveted-steel of $\frac{7}{16}$-inch plate, while the newer line consists of a 76-inch welded-steel pipe of $\frac{1}{2}$-inch plate. Expansion joints are provided over every other bridge pier, the pipes being anchored at the other or alternate piers.

Crossing the marsh, on both sides of the bay, each pipe line is

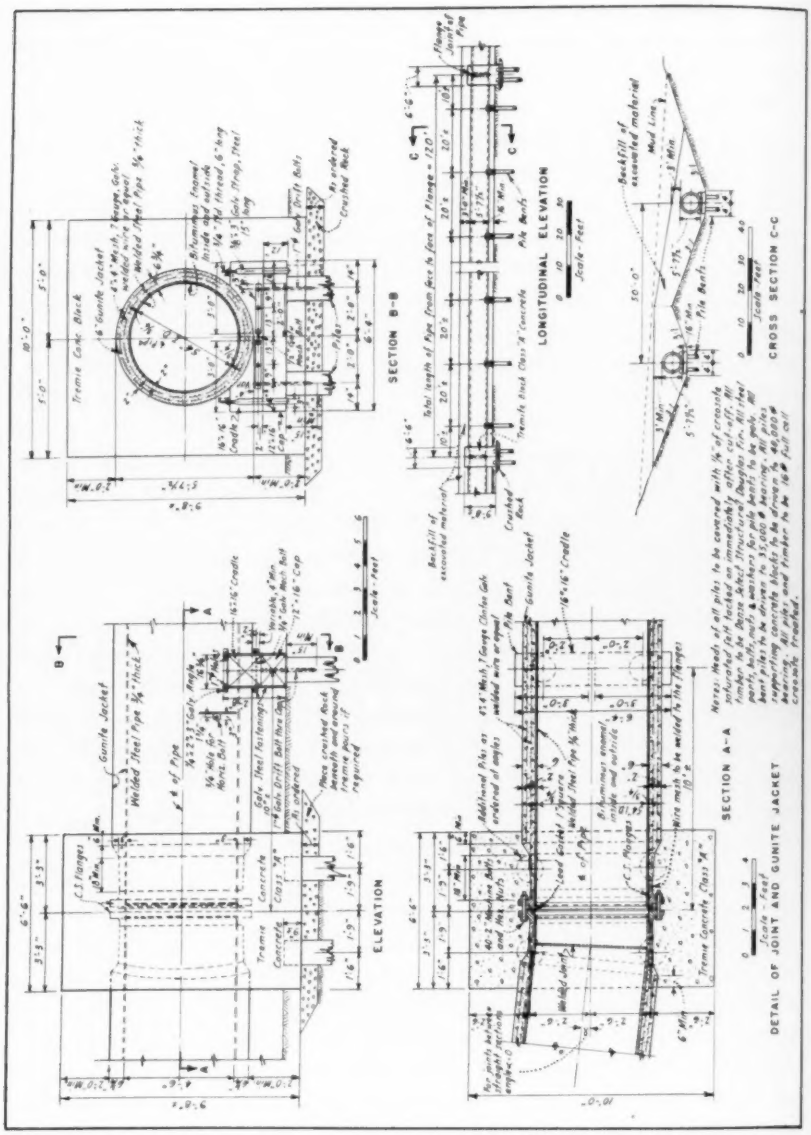

FIG. 10. Bay Crossing Pipe Line, Submarine Section: typical details for the twin 54-inch pipes and supports.

supported on timber trestles aggregating 4 mi. in length. Near the west shore of the bay is located the Pulgas pump station which was constructed, in connection with the first bay crossing pipe of the Hetch Hetchy project under agreement with the Spring Valley Company, to boost water released from Calaveras into Crystal Springs reservoir. This station is now used in emergencies to handle water from Sunol delivered to it through the old Alameda line. Entering Pulgas tunnel, elevation 290, at the westerly end of these two

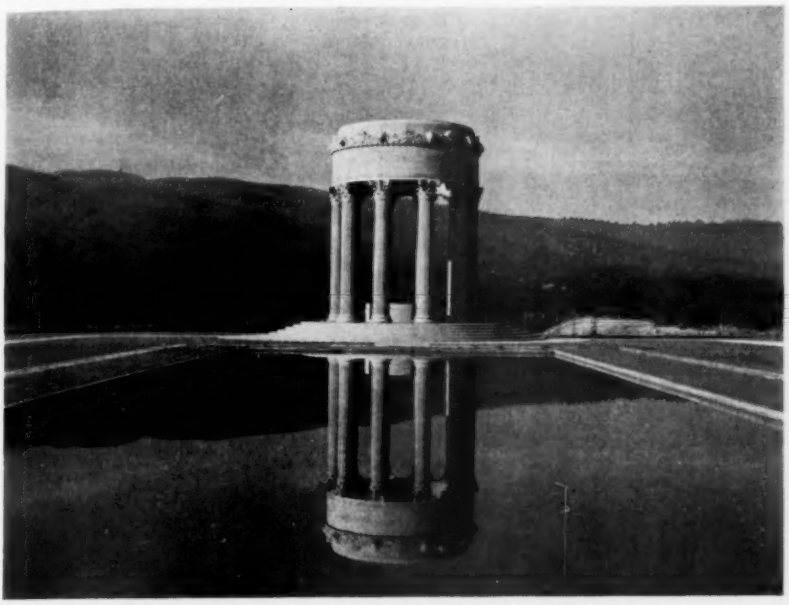

FIG. 11. Pulgas Water Temple: This classic structure with reflecting pool marks the westerly end of the 155-mile Hetch Hetchy aqueduct. The surrounding area is being landscaped to provide a small park open to the public.

pipe lines, the water flows through $1\frac{3}{4}$ mi. of concrete lined tunnel 10 ft. 3 in. in diameter and is discharged through an outfall canal into Crystal Springs reservoir. Water from Hetch Hetchy reservoir or Calaveras is thus delivered to Crystal Springs entirely by gravity. Water from the other Alameda sources must be boosted. To mark the terminus of this aqueduct an ornamental temple of cast stone and concrete has recently been completed. Enscribed on the entablature is the scriptural quotation: "I give waters in the wil-

TABLE 1

San Francisco Water Supply Dams

NAME	TYPE	YEAR		MAX. HEIGHT ABOVE FOUND.	CREST		SLOPES		CONTENTS
		Built	Raised		Length	Elev.	Upstream	Downstream	
				feet	*feet*	*feet*			*cu.yd.*
Pilarcitos	Earth	1866	1874	95	520	700	2½:1	2¼:1	371,200
San Andres	Earth	1870	1875, 1928	105	950	456	3½:1 & 2:1	3:1 & 1½:1	540,000
Upper Crystal Springs	Earth	1877	1891	92.5	520	288	3¼:1 & 1½:1	3½:1 & 2:1	220,140
Lower Crystal Springs	Concrete-Gravity	1888	1890, 1911	154	600	288	—	—	157,200
Calaveras	Earth	1925	—	220	1200	775	3:1 & 5:1	Ave. 2¾:1	3,461,280
O'Shaughnessy	Concrete-Gr. Arch	1923	1938	430	900	3812	—	—	675,000
Eleanor	Conc.-Buttressed Arch	1918	—	70	1260	4661	—	—	11,640
Upper Alameda Diversion	Concrete-Slab and Buttress	1932	—	31	173	900	—	—	4,500
Early Intake Diversion	Concrete-Arch	1918	—	81	262	2356	—	—	15,560
Priest Regulating	Earth and Rock	1923	—	160	1160	2245	2½:1	2:1 & 1¾:1	717,280
Moccasin Re-regulating	Earth and Rock	1931	—	70	855	927	2½:1	2:1	143,340

derness and rivers in the desert, to give drink to my people." (Isa. 43:20.)

The essential information presented in this description of the dams and reservoirs which make up the source of the San Francisco Water Supply are shown in Tables 1 and 2.

The City Distribution System

The transmission mains from the Peninsula reservoirs, as previously noted, discharge into certain distributing reservoirs within the city which we will call terminal reservoirs as distinguished from the secondary distributing reservoirs and tanks within the city to which

TABLE 2

San Francisco Water Supply Storage Reservoirs

NAME	ELEVATION HIGH WATER	AREA	CAPACITY	DAILY YIELD	WATERSHED-AREA	
					Direct Drainage	Contributing
	feet	*acres*	*mil.gal.*	*mil.gal.*	*sq.mi.*	*sq.mi.*
Pilarcitos	695	109	1,010	16	3.8	1.4
San Andres	446	550	6,190		4.4	4.0*
Upper Crystal Springs	288	1,492	22,580		9.0	
Lower Crystal Springs					9.5	—
Calaveras	752.5	1,435	31,550	30**	100.0	35.0
Hetch Hetchy	3,806.	1,960	117,300	180***	459.0	—
Lake Eleanor	4,661.	945	9,000		79.0	—
			187,630			

* Can contribute to either San Andres or Lower Crystal Springs.

** Including diversion from 35 sq.mi. on Upper Alameda Creek.

*** Amount of present day yield available for water supply.

water from these terminal reservoirs may be delivered either by gravity or pumping. There are five such terminal reservoirs in San Francisco: Lake Honda, College Hill, University Mound, Sunset, and Merced-Manor; these have a total capacity of nearly 300 mil.gal.

Lake Honda, the oldest of these, was constructed in 1861 to receive water from Pilarcitos Creek. This reservoir which is not covered is located near the geographical center of the city and occupies a natural depression, the capacity of which was increased by the construction of massive masonry walls of stone, brick or concrete on one side. All natural slopes are lined with concrete

approximately 12 in. thick; the floor is also lined with concrete. In 1915, the capacity of the reservoir was increased to the present figure of 44 mil.gal. by the construction of a 6-foot retaining wall around its perimeter. Water is carried in the reservoir to a depth of 35 ft. With the completion of the Sunset reservoir Lake Honda will be used to provide auxiliary storage and for emergency operations.

FIG. 12. Sunset Reservoir under Construction: Capacity 89.4 mil. gal. at elevation 385.5 is supplied by gravity from San Andres reservoir and with Lake Honda serves approximately one-third the built-up area of San Francisco. The roof area approximates 12 acres.

The new Sunset reservoir, which was completed in 1938, supplants Lake Honda for normal operating purposes. This is due to several reasons: the new reservoir serves the same district, it is covered, and its water surface is 12.5 ft. higher than that of Lake Honda. In spite of the additional height, delivery can be made by gravity from San Andres at a higher rate owing to the existence of a high point in the profile of the transmission line just before reaching Lake Honda. The Sunset reservoir is located in the western part

of the city and covers over four large city blocks. It has a storage capacity of 89.4 mil.gal. and with Lake Honda will make available five days' supply to the rapidly developing residential district which lies in the area served. With the reservoir full the average water depth is 29.5 ft. The reservoir is formed by excavation and embankment, the outer slopes of the embankment being $2\frac{1}{2}$ to 1 and the inner slopes 3 to 1; where the reservoir is in excavation the inner slopes are $2\frac{1}{2}$ to 1. The reservoir basin, which is largely in sand formation, is lined with a clay blanket 2 ft. thick over which 6 in. of reinforced concrete is placed. The reservoir is covered with a roof of reinforced concrete having an area of nearly 12 acres.

Merced-Manor reservoir located in the southwestern part of the city was completed in 1936. This reservoir has a capacity of 9.5 mil.gal. and replaces a small concrete reservoir of 0.5 mil.gal. which was located at the same place. Serving as the terminal reservoir for the 30-inch Baden-Merced pipe line, this reservoir also supplies consumers in the lower levels along the ocean. Water is taken directly from the reservoir by the central pumping station adjacent to it and elevated to the Stanford Heights reservoir. Slopes of the earth embankment for the reservoir are $1\frac{1}{2}$ to 1 inside and 3 to 1 outside. Like Sunset, it is concrete lined and roofed, the roof area being 2 acres. Due to its location in the heart of a residential area the entire reservoir tract has been planted with lawn and landscaped to present a pleasing aspect.

College Hill reservoir, next in point of age to Lake Honda, is located in the Mission District. It was constructed in 1870 to receive water from San Andres reservoir. The capacity of the reservoir is 13.5 mil.gal. at high water level of elevation 255. The depth of water is 16.5 ft. This reservoir was constructed by excavation and earth embankment with a clay lining. Inside slopes are about $1\frac{1}{2}$ to 1 and are faced with uncemented rubble-rock masonry. It is an open reservoir and due to its shallow depth is subject to heavy weed growths necessitating annual cleaning.

University Mound reservoir, in the southeast part of San Francisco, consists of two basins, one of which was constructed in 1885 and enlarged in 1924, and the other completed in 1937. The newer basin, covering six city blocks, has a storage capacity of 81.5 mil.gal., and the older 59.4 mil.gal. The old basin has an average water depth of 26.5 ft.; the new 25.3 ft., high water level for both being elevation 172. With the completion of this new basin there is

available six days' supply for the district served, which includes most of the highly congested downtown mercantile and commercial

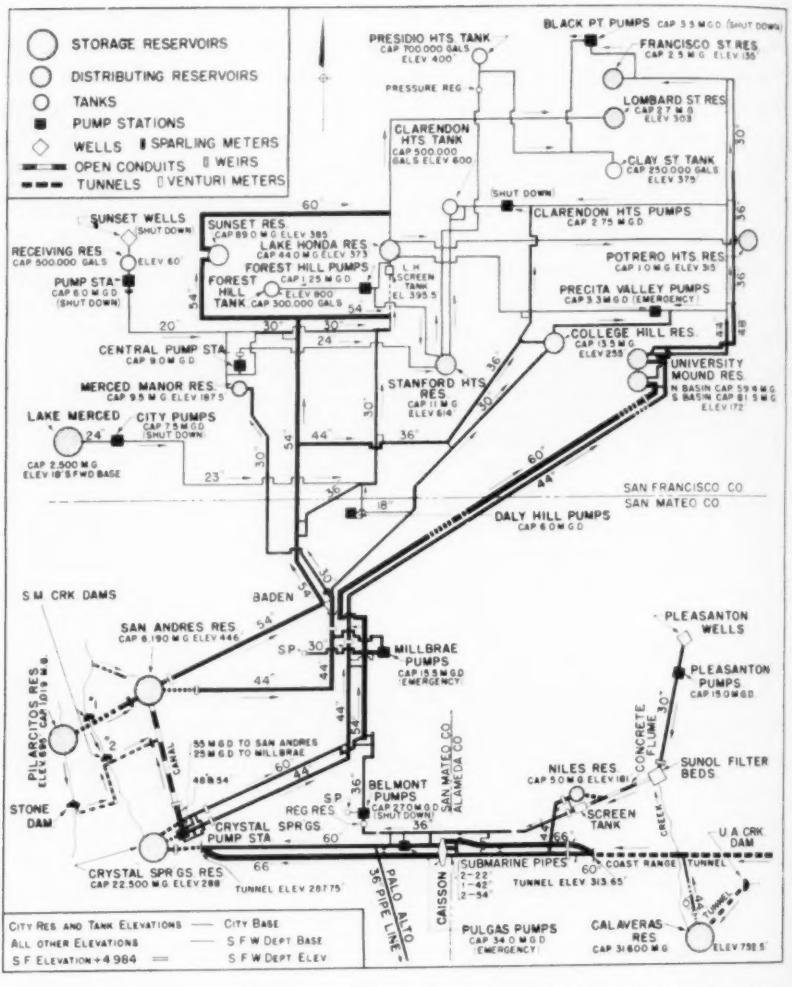

FIG. 13. Diagram of System: Shows diagrammatically relative location and elevations of the major elements of the system and the connecting conduits.

areas and extends along the entire water front to the Presidio. Both basins were constructed in excavation and earth embankment. The older basin has inside and outside slopes of 3 to 1 and is lined

with 6 in. of concrete which rests on a 12-inch layer of puddled clay. The raising of this basin in 1924 was accomplished by adding to the embankment and lining the inner slope of the raised section with 2 in. of gunite. Slopes of the raised portion are partly 1½ to 1 and partly 2 to 1 inside; outer slopes are 2 to 1. Slopes of the new basin are 1½ to 1 inside and 3 to 1 outside. The entire basin of the newer reservoir is lined with 6 in. of reinforced concrete and covered with a reinforced-concrete roof having an area of 11 acres.

The three new reservoirs, University Mound, Sunset and Merced-Manor, are all essentially similar in design in that the lining which is 6 in. thick in all cases is reinforced with ½-inch deformed round bars spaced 8 in. in both directions. Construction joints are 25-foot centers except in Merced-Manor where they are 21.25 ft. The roofs consist of 2½-inch reinforced-concrete slabs supported on reinforced-concrete joists and girders all cast integrally. The supporting columns are circular, in most cases 24 in. in diameter.

The roofs on the two larger reservoirs are designed for live loads of 100 lb. per sq.ft., Merced-Manor being designed for 150 lb. All are designed as earthquake resistant structures, on the assumption of a horizontal force equal to 10 per cent of gravity; temperature stresses are also provided for. The roofs have expansion joints which are spaced approximately 250 ft. apart.

Supply for Treasure Island

Water for Treasure Island is supplied by the San Francisco Water Department, and this has contributed in no small part to the beauty of the Golden Gate International Exposition, for without a plentiful supply of good water the soil could not have been properly prepared and the magnificent landscaping could not have been possible. The Fair management estimated that a maximum demand of 2.5 m.g.d. would be required, not including possible fire demand which could be pumped from the bay into a separate system.

Water from San Francisco's University Mound District is pumped to a 3 mil.gal. reservoir on Treasure Island through a 10-inch steel pipe line 10,000 ft. in length supported under the upper deck of the San Francisco-Oakland Bridge, and 1,300 ft. of 12-inch cast-iron pipe on the Island. The pump station is located at the No. 1 bridge pier on the Embarcadero and is equipped with two 175 g.p.m. electric-driven pumping units, one 1,250 g.p.m. unit, and one 1,750 g.p.m. unit. When supplying the maximum estimated amount of

2.5 m.g.d., the static lift, plus friction losses, is 382 ft. Motors for these pumps are 10 h.p., 125 h.p. and 250 h.p., respectively. The two small units are intended, primarily, to supply the permanent needs of the Navy on Yerba Buena Island and an airport to be built by the city on Treasure Island, when the Fair is removed. The 1,250 g.p.m. pump is used for normal water consumption and to meet the fire protection needs of the bay bridge, and the largest

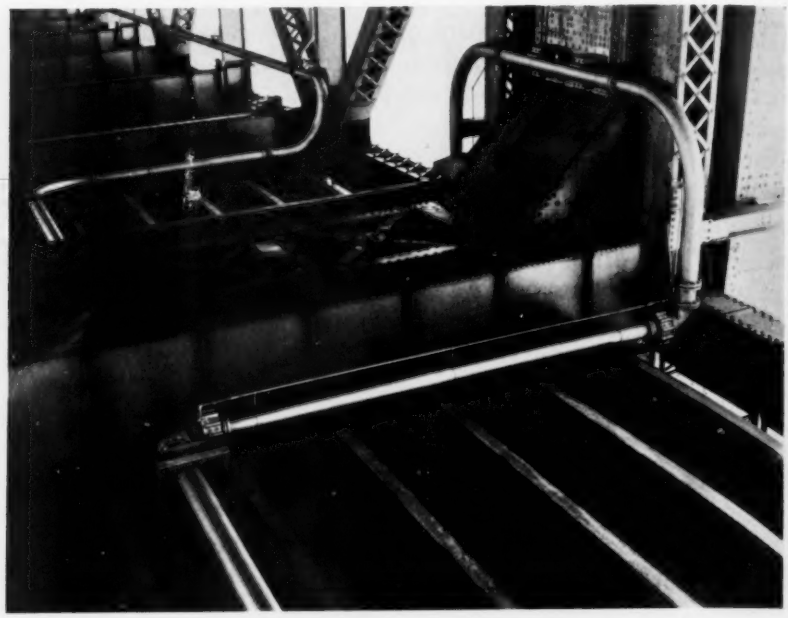

FIG. 14. Bay Bridge Pipe Line: Serves Treasure Island. View shows group of ball and socket joints installed at bridge piers to provide for horizontal, vertical and longitudinal movement of the supporting bridge structure.

unit, 1,750 g.p.m., is used only on days of peak consumption at the Exposition.

Some unique problems were met when planning the support for the pipe on the bridge and at the same time providing for the various longitudinal, horizontal, and vertical movements of the supporting bridge members due to temperature, wind and loading. To accommodate this movement, the maximum of which was 30 in. in a longitudinal direction, groups of four 8-inch inside-diameter spherical joints were provided at the central anchorage and at each of the four mian towers in the bay. At each end of the pipe line on the

bridge two such joints were provided. These joints were specially designed by department engineers for this purpose and these in some respects are unique. Each joint can deflect a maximum of 8 degrees and being spaced some 25 ft. apart, in a transverse direction across the bridge, they permit the desired movement. Spherical joints are made tight by specially designed rubber rings, backed on the inside with reinforced canvas and on the outside with a bronze band which prevent pinching the rubber. Internal pressure tends to close the joint between the rubber ring and the housing.

Along the bridge at intervals of from 240 ft. to 420 ft. are expansion joints on one side of which the pipe is anchored and on the other side free. These joints provide for differential expansion between various sections of the bridge.

All spherical joints and expansion joints are designed for 250 lb. per sq.in. pressure and are so arranged that any leakage which may develop will be collected and conveyed away from the bridge floor.

At Yerba Buena Island the 10-inch steel pipe connects with the 12-inch cast-iron main which skirts the Island and delivers the water to the 3 mil.gal. reservoir whose high water level is elevation 260. This reservoir located on a side hill is constructed mostly in excavation lined on the slopes with gunite 4 in. thick. On the lower side of the hill, however, a 12-foot concrete retaining wall was necessary. The reservoir is covered with a wood roof supported on a steel substructure.

From the reservoir a 22-inch steel main leads 1,800 ft. to Treasure Island where it branches into a 16-inch for high pressure fire service, and an 8-inch for domestic and sprinkler uses. A pressure reducing valve in the 8-inch reduced the pressure from 110 to 60 lb. per sq.in. The domestic system totaling 7.3 mi. branches out from an 8-inch to 6-inch, 4-inch and 2½-inch mains, and the high pressure system totaling 9.7 mi. reduces from 16-inch to 12-, 10-, 8- and 6-inch.

The pumping station and pipe line on the bridge were designed by the San Francisco Water Department and the construction work carried out, under contract, by the U. S. Army engineers. Work on the reservoir was carried out by Works Progress Administration forces, under direction of the United States Army engineers. The domestic and high pressure system on Treasure Island, including the 22-inch pipe from the reservoir, was under the jurisdiction of the engineers of the Golden Gate International Exposition.

Pressure Districts

The distribution system in San Francisco is necessarily somewhat complicated owing to the many hills which rise more or less separately and in many cases rather abruptly in different locations throughout the city at an elevation range from sea-level to 900 ft. To provide satisfactory pressure conditions, what may be considered as five major pressure zones have been established within the limitations of the available sites for, and the elevations of, the reservoirs or tanks serving them. However, due to the uneven topography of the city these zones are broken up, particularly in the higher elevations, into a number of isolated areas, so that there are some twenty-five different pressure districts established throughout the city by closed gates between different parts of the pipe system. These districts vary greatly in outline and area and frequently can be served from more than one source. In the earlier history of the system there were but three districts: Lake Honda, College Hill and University Mound, each of which obtained its supply from one of the main lines entering the city from the Peninsula sources. With the growth of San Francisco, and the building up of outlying districts, it became necessary to add new pressure districts, and provide additional reservoirs, tanks, pumps, and feeder mains to supply them. Frequently, the boundaries of previously established districts have been changed. The changes and additions made from time to time to serve an immediate need tend to become perpetuated and add to the complications imposed by natural conditions.

The four largest districts, namely, Lake Honda (or Sunset), University Mound, College Hill and Stanford Heights, serve about 75 per cent of the closely built area of the city and are supplied by gravity from the correspondingly named reservoirs. Three districts near the southern limits of the city are supplied directly from the transmission mains before reaching the reservoirs. Five districts at higher elevations are supplied by pumping from the terminal reservoir and five districts are served with the proper pressure by reducing valves or friction losses in small pipes.

University Mound reservoir, with a total capacity of 131 mil.gal., at elevation 172, supplies by gravity the lowest pressure zone, which comprises in general the waterfront, industrial, and major business areas. This district borders on San Francisco Bay and lies below the 90-foot contour. It comprises about 25 per cent of the built-up area of the city. A portion of the district near the county line is

supplied directly by the transmission lines from Crystal Springs before reaching University Mound reservoir. Francisco Street reservoir (at elevation 135), which was built in 1861 and which has a capacity of 2.5 mil.gal., is connected into the University Mound system and acts as an equalizing reservoir.

The College Hill pressure district includes about $\frac{1}{10}$ of the built-up area lying west of the University Mound District with elevations ranging between 55 and 230 ft. It takes in a small part of the congested business district and some mercantile districts but it is chiefly a closely built residential area. This district is supplied by gravity from the College Hill reservoir with a capacity of 13.5 mil.gal. The 30-inch transmission line from San Andres reservoir supplies an isolated portion of this district southwest from the reservoir.

The Lake Honda pressure district, which may presently be identified as the Sunset district, supplies $\frac{1}{3}$ of the built-up area of San Francisco. It lies mostly in the northwestern part of the city with extensions to the east and southeast and lies generally between elevations 50 and 330. This district includes some of the business area but is mostly residential. The supply is by gravity from Lake Honda, capacity 44 mil.gal., or from Sunset reservoir, capacity 89.4 mil.gal. In emergencies service can be provided to a limited extent by pumping from the Sunset Wells, Lake Merced or College Hill and University Mound districts. The Lake Honda district also includes some areas adjacent to the San Andres transmission mains that are fed directly therefrom.

Stanford Heights pressure district supplies a moderate area of irregular shape, mostly on the hillsides near the geographic center of the city. This district lies between the 265- and 500-foot contour. It is almost entirely residential in character. A portion of the Stanford district is supplied from the reservoir of the same name and a portion directly from the force main leading to the reservoir from Central pumping station. This pump station takes water from either the 54-inch San Andres line or from Merced-Manor reservoir.

Supplied from the Stanford Heights district through pressure-regulating valves is the Presidio Heights district, a small area on three hills in the northern part of the city. Elevation ranges from 185 to 350 ft. It is solidly built up of residences and apartments.

Forest Hill district comprises a small area of detached residences south and west of Lake Honda lying between the 525- and 710-foot

contour. It comprises the highest of the major districts in the city, and is supplied by pumping from Lake Honda with the Forest Hill tank (elevation 800, capacity 300,000 gal.) as an equalizer.

Merced-Manor reservoir supplies a district known as Oceanside, adjoining the Pacific Ocean and lying below the 90-foot contour. It is mostly scattered residences. This district, while under pressure limits similar to University Mound is entirely separated therefrom by intervening hills.

Aside from the terminal reservoirs, having a capacity of 300 mil.gal., the distributing system includes five other reservoirs: Francisco Street, capacity 2.5 mil.gal.; Lombard Street, capacity 2.7 mil.gal.; Potrero Heights, capacity 1.0 mil.gal.; Stanford Heights, capacity 11.0 mil.gal.; and Wilde Avenue, capacity 0.5 mil.gal. Elevations and data relative to these are shown in Table 3. In addition to these there are 15 tanks varying in capacity from 7,000 to 700,000 gal. and having a total capacity of approximately 3.5 mil.gal.

Pumping Stations

Although the installed pumping capacity within the city is some 37 m.g.d., most of these units are no longer in normal service and aside from a number of small electric units having a total capacity of less than 1.0 m.g.d., the only pump stations at present regularly in use are the central pumps, with a capacity of 9.0 mil. gal., two 4.5 m.g.d. steam-turbine driven units, and the electric-driven Forest Hill Station with a capacity of 1.2 m.g.d.

The San Francisco distribution pipe system comprises a total of about 950 mi. of pipe, not including transmission mains, or hydrant connections. Approximately 160 mi. of this is wrought-iron or steel pipe varying in size from $\frac{1}{2}$ in. to 60 in. in diameter, less than 40 mi. being under 2 in. in diameter. The balance is cast-iron pipe varying in size from 2 to 30 in., only 13 mi. of which are less than 4 in. in diameter. At present no pipe under 6 in. in diameter is being laid, except in a few minor instances where fire service is not involved.

The entire system is well gridironed, the average length of 4- and 6-inch mains between cross-connections in the long side of the closely built blocks is 530 ft. In general, it is the policy to reserve main arteries for the supply of secondary feeder pipes and to connect minor distribution only to the latter.

The average age of the steel and wrought-iron pipe is 16 years and the oldest is 59 years. The average age of the cast-iron pipe is 33

and the oldest 80 years. Present practice is to use cast-iron pipe for sizes up to 16 in.; for larger sizes welded steel pipe is used.

Approximately 8,000 gate valves larger than 4 in. are installed in the system, 70 per cent of which are 6 in. and 8 in. and the largest 48 in.

San Francisco's low per capita use of water, about 72 gal. per day, is largely due to high density of population (17,000 per sq.mi.), the small lots and closely built-up residential areas and universal use of meters of which there are approximately 118,000 in service.

Characteristics and Quality of the Water

From the standpoint of freedom from pollution, all San Francisco's water sources are exceptionally good due to supervision and patrol of watersheds, a large portion of which area is owned by the city, or which as in the case of Hetch Hetchy is in national park or forest lands. Habitation on these watershed areas is less than one person per square mile and in most instances these are caretakers or other employees. Residences of all employees on the Peninsula watersheds are equipped with concrete receiving tanks for sewage and waste material which is hauled off the watershed for disposal.

Records of bacteriological examination of raw waters from the storage reservoirs indicate from 12 to 15 per cent of the 10 c.c. portions as positive for coliform organisms. Chlorination and ammoniation treatment reduce this percentage to from 0 to 1.4, well under the 10 per cent allowable under federal requirements for water for interstate carriers.

Chemically, the water varies greatly depending upon the source, the rainfall and the runoff characteristics. Hetch Hetchy water is very soft, containing almost no mineral matter. The hardness of this supply varies from 6 to 12 parts per million. This water contains 1 p.p.m. chloride, 5 to 15 p.p.m. alkali and has a hydrogen ion value of from 6.4 to 7.2. Water produced by the Peninsula reservoirs and Calaveras is considerably harder, running between 80 and 175 p.p.m. hardness. The water as delivered to consumers is a combination of Hetch Hetchy and local waters and varies from 60 to 120 p.p.m., depending upon the proportion of each in the mixture. At the present time the water served in the city has a hardness of 74 p.p.m., chlorides, 8 p.p.m.; alkalinity, 60 p.p.m.; and a pH of 7.7. Physically, the water is good; the turbidity ranges

from 0 to 5 p.p.m. and averages about 2 p.p.m. Normally, there are no objectionable tastes or odors although on occasions troublesome algae growths and weed growths in the reservoirs have caused complaints.

Sanitary Control

Natural and artificial agencies for sanitary treatment may be classed under four headings: (1) watershed sanitation; (2) storage; (3) disinfection; (4) laboratory control.

TABLE 3

Distributing Reservoirs in San Francisco

NAME	YEAR BUILT	DEPTH OF WATER	ELEVATION HIGH WATER	AREA	CAPACITY
		feet	*feet*	*acres*	*mil.gal.*
Lake Honda	1861 & 1915	35.0	373	6.0	44.0
College Hill	1870	16.5	255	3.1	13.5
University Mound (Old)	1885 & 1924	26.4	172	9.8	59.4
University Mound (New)	1937	25.0	172	10.5	81.5
Sunset	1938	30.0	385	11.5	89.4
Merced Manor	1936	20.5	187.5	1.7	9.5
Stanford Heights	1923 & 1928	20.2	614	2.5	11.0
Francisco Street	1861	8.0	135	1.7	2.5
Lombard Street	1860	17.5	303	0.8	2.7
Potrero Heights	1897	21.5	315	0.3	1.0
Wilde Avenue	1902	10.0	400	0.15	0.5
Total					315.0
Lake Merced	1895	30.0	18	386	2,500.0

In addition to the careful control of sewage from the caretakers and employee's residences, watershed sanitation measures include ownership of lands, fencing, and regular patrolling of the drainage areas. All watersheds are patrolled by rangers and reservoir caretakers of which there are at least two at each storage reservoir. A few well-traveled roads traverse the watersheds but there are no camping grounds or resorts anywhere on these lands. Access to the watershed lands is by permit only. Picnicking by small groups is permissible in only a few places which are under careful supervision and regulation. Peninsula and Calaveras watershed lands, under city ownership, are entirely fenced. Privately-owned property on the watershed lands is supervised for sanitation by the water de-

partment with the cooperation of the State Department of Public Health.

Storage acts as a natural agency for the self-purification of the water supply and the San Francisco impounding reservoirs are of sufficient size to give the water a long period of storage before serving to the consumers. Fishing, bathing, boating, or other recreational activities in the reservoirs or marginal lands are entirely prohibited.

TABLE 4

Typical Mineral Analyses of San Francisco Water
(Results expressed in p.p.m. except pH and conductivity)

SOURCE	CRYSTAL SPRINGS*	HETCH HETCHY RESERVOIR	CRYSTAL SPRINGS**
Date Sampled	7-23-34	8-27-36	9-5-39
Conductivity	38.5	1.97	16.9
pH	7.6	6.4	7.7
Dissolved Solids	195	14	107
Silica & Insolubles	13	3.8	4.0
Sodium (Na)	19	0	7.65
Calcium (Ca)	39	1.1	17.5
Magnesium (Mg)	16	1.4	6.39
Iron (Fe)	0.02	0.025	0
Aluminum (Al)	3	0.56	0.6
Bicarbonate (HCO_3)	177	6	73
Sulfate (SO_4)	28	1.6	15.5
Chloride (Cl)	21	1	8
Hardness as $CaCO_3$	163	11	74
Alkalinity as $CaCO_3$	145	5	60

* Before advent of Hetch Hetchy water.

** Represents water supplied to San Francisco at present.

Disinfecting the water supply is accomplished by chlorine and ammonia applied as the water is withdrawn from storage reservoirs. In some cases where the water passes through open distributing reservoirs the supply is again chlorinated. Dosages average 0.3 p.p.m. with an ammonia-chlorine ratio of 1 to 5. Chlorine and ammonia are applied also at the upstream end of the Coast Range tunnels, at the Tesla chlorinating plant, in sufficient quantities to carry a residual of 0.2 p.p.m. throughout the 28-mile tunnel. Chlorine stations are visited in most cases twice daily by operators trained to make residual tests and keep the necessary records. All equip-

ment is kept in first class condition and at least two machines are located in each plant. The department also maintains a portable emergency chlorinating unit which consists of a vacuum type chlorinating machine and a gasoline-engine-operated pump mounted on a trailer, which unit can be taken to any part of the system for use in disinfecting mains or for any emergency use.

A complete laboratory and staff of purification engineers exercise control over water treatment for the system. Over 100 samples are analyzed weekly including bacteriological examinations and tests for specific electrical conductance, hardness, chlorides, alkalinities, pH value, and turbidities, all of which are determined for practically every sample. Complete mineral analyses are run of the sources and reservoirs each three months. Twice a week samples from each reservoir are analyzed for plankton growths. To control algal growths, copper sulfate treatments are applied to the reservoirs. Normally three dosages, averaging 1 lb. of copper sulfate per mil.gal. of water stored, are required each year. An improved method of applying copper sulfate has been developed by the department by which the chemical is applied mixed with water by means of a spray over the reservoir surface, insuring uniform application.

Frequent studies of the condition of the water in the various reservoirs are made to determine chemical and physical qualities of the water. This work is done so that water may be withdrawn from the reservoirs at the outlet best situated to give good water. Results of typical mineral analyses are shown in Table 4.

Summarizing, San Francisco has an abundant supply of water which is of satisfactory chemical and physical characteristics and which is universally safe from a disease standpoint. So far as is known, the water supply has never been responsible for a single case of typhoid fever. The water is relatively soft and satisfactory for industrial and domestic consumption. Treatment of the supply is continuous and is carefully supervised by means of a well-equipped laboratory and staff.

Depreciation of Cast-Iron Water Mains Related to Actual Service Lives

By Reeves Newsom and E. H. Aldrich

FURTHER study of the data regarding service lives of water mains as developed from the records of the Springfield, Mass., water system reported in the JOURNAL for October, 1939, throws some light on the questions raised in that paper.

When prescribing methods of accounting for depreciation, public bodies having jurisdiction over water as well as other types of utilities have rarely given much weight to the basic differences involved, since service lives of a major portion of the property of water utilities are of the order of one or more hundreds of years as contrasted with the 10 to 25 years of life in the case of electric and telephone utilities. This is undoubtedly due to the fact that water utilities occupy a relatively minor place in the work of the boards and commissions. As a result, depreciation of water property has been considered largely in the light of the experiences of the types of utilities which are more numerous.

Depreciation accounting as such has not been common with municipal water utilities. The trend is definite, however, toward proper accounting on a uniform basis for both publicly and privately owned plants and the problem is one in which publicly owned works have a vital interest. In the past the amortization of bond issues has been considered to some degree a substitute for a provision for depreciation. Because of the failure to give effect to the long service lives of cast-iron pipe, bond issues frequently provide for amortization in 30 years when the property they cover will continue in service many times that long. As a result the consumers of one generation are paying for the retirement of property which succeeding generations will use free of such a charge.

A paper presented on April 22, 1940, at the Kansas City Convention by Reeves Newsom and E. H. Aldrich, Engineer-Consultants, New York.

The history of retirements of pipe at Springfield, Mass., during the 76 years covered by the system's records indicates, as noted in the previous report, an average life of about 50 years for 4-inch pipe, materially in excess of 100 years for 6-inch pipe and correspondingly longer lives for larger sizes.

In attempting to account for the depreciation of water works property, particularly for long-lived elements such as cast-iron water mains, the shortcomings of the straight-line computation method are greatly magnified. In view of the fact that main pipes represent such a substantial part of the investment in a water works plant, the resulting errors are of importance.

The claims made by those favoring the straight-line computation method are that it represents the simplest method of spreading the cost of retirements over the life of the property; that the so-called sinking-fund method is too difficult of application; that the use of the observed-depreciation method makes no allowance for many factors working toward the ultimate retirement of the property which are not physically evident and that, in any event, in the case of water mains there is no practical way to make sufficient and adequate observations.

Those favoring expert opinion based upon observation as a means of determining accrued depreciation contend that it meets the objection of the courts to arbitrary computations; that as long as an element of property is giving adequate service there is no evidence as to its future retirement other than its physical condition; that any allowance for future functional depreciation must, in the last analysis, be a judgment figure; that the straight-line method of computing the depreciation-reserve requirement results in an amount many times greater than is needed; and that in those circumstances where the reserve requirement is considered in arriving at a rate base or a purchase price its excessive size results in confiscation of property.

This paper is not a discussion of the relative merits of age-life computations and of expert opinion based upon observed condition as a means of determining depreciation. Its purpose rather is to demonstrate that if age-life computations are used, a method simple enough to be practical can be related closely to the way in which water mains will be retired and yet have the resulting depreciation reserve at any time represent a reasonable allowance beyond the amount required for near future retirements to cover the progress toward ultimate retirement of the pipe remaining in service.

Depreciation Defined

Because the subject of depreciation is of such importance in water works management and because so many misconceptions exist about its true nature, a brief review of its fundamental theory may be well worth while.

The definition of depreciation from an accounting standpoint has become fairly well standardized by such boards as the Interstate Commerce Commission, National Association of Railway and Utility Commissioners, Federal Power Commission and many state utility commissions in connection with the promulgation of classifications of accounts for the various utilities under their respective jurisdiction. The most generally accepted definition is as follows:

"Depreciation, as applied to depreciable utility plant, is defined as the loss of service value not restored by current maintenance incurred in connection with the consumption or prospective retirement of utility plant in the course of service from causes which are known to be in current operation and against which the utility is not protected by insurance."

Several important factors in this definition deserve to be pointed out. Depreciation affects only depreciable plant. Thus land and other property which have some remaining value upon retirement from service as evidenced by salvage, are not subject to complete depreciation.

The definition says that depreciation is the loss of service value; that is, the ability to render service and to earn a return upon such service measures at any time the value of plant, and the difference between such value new and at any subsequent time is the existing depreciation. Obviously, therefore, if the plant has some remaining service value it is not fully depreciated and cannot be eliminated from further consideration simply because it has reached some arbitrarily selected age. It is the future service which that plant can render which must be determined before the proportion of loss can be found. The definition further recognizes that property depreciates only when its service value is not restored by maintenance. The basic element of the definition is that depreciation is the consumption or prospective retirement of utility plant. Any cause or factor consuming the property or tending to 'force' its retirement is causing depreciation.

Under this definition, therefore, as applicable to cast-iron pipe,

bearing in mind that except in rare instances pipe is never consumed in service, depreciation is almost entirely dependent upon how long such pipe remains in place rendering service. The prevalent notion that tuberculation, by decreasing the carrying capacity of pipe, is a factor causing depreciation is not consistent with the definition if such reduced capacity can be corrected by maintenance. The existence of such a condition perhaps may be the result of deferred maintenance temporarily reducing service capacity, but basically it is in the nature of maintenance and not of depreciation. In those exceptional cases where the corrosion of pipe is such as to consume it and hasten its prospective retirement it is properly depreciation.

Nothing in the definition of depreciation indicates how to determine at what rate the loss of service value takes place. This has been the subject of various interpretations, important as the cause of disagreement among engineers and accountants dealing with these problems. From the definition, however, theoretical considerations of factors causing depreciation upon other bases than the prospective retirement of pipe have been ruled out by most commissions.

No method of computing or accounting for depreciation can be correct unless there is known, concerning the group of units of property under consideration, the average life, the life span of the longest survivor and the variation in the rate of disappearance from service of the units of property during that life span. As it is obvious that such data cannot be definitely determined until the end of the service life of such property, it is essential that, if a computation method is to be employed, the lives used in determining depreciation accruals should be estimated as accurately as possible in order to reduce to a minimum the final adjustment necessary in accounting for the consumption of such property in service. To that end the use of such long term information regarding survival of pipe in service as is or can be made available is greatly to be preferred to arbitrary assumptions or estimates based on meager data of partial early retirements.

Applicability of Mortality Data

Many commissions have prescribed regulations for the installation and maintenance of continuing property records, one of the main purposes of which is to determine the actual service lives of various utility property. In the case of cast-iron pipe with its long span of service life, however, it will be many years before records started at this time will be of any value in determining the average service life of pipe in each plant. Because of this fact, dependence must be

placed on the knowledge that can be gained by going backwards into the records of such plants as have them available for a long period.

TABLE 1

Mortality Data of Cast Iron Pipe—Springfield, Mass.

	UNIT	DATA AS OF	
		1938	1939*
4-inch Cast-Iron Mains			
Total pipe placed	ft.	74,990	80,851
Total pipe retired	ft.	33,409	37,322
Average retirement rate	%	1.151	1.179
Survival, total record	%	20.707	14.743
Survival, 10-year record	%	—	7.022
Length of record	yr.	68	69
6-inch Cast-Iron Mains			
Total pipe placed	ft.	491,599	525,298
Total pipe retired	ft.	43,417	44,845
Average retirement rate	%	0.232	0.223
Survival, total record	%	89.240	89.188
Survival, 10-year record	%	—	94.466
Length of record	yr.	72†	73
8-inch Cast-Iron Mains			
Total pipe placed	ft.	604,047	639,742
Total pipe retired	ft.	19,966	20,077
Average retirement rate	%	0.154	0.143
Survival, total record	%	89.788	90.484
Survival, 10-year record	%	—	95.405
Length of record	yr.	72†	73
10-inch and Larger Cast-Iron Mains			
Total pipe placed	ft.	392,731	404,171
Total pipe retired	ft.	14,326	14,326
Average retirement rate	%	0.121	0.116
Survival, total record	%	92.898	93.155
Survival, 10-year record	%	—	99.313
Length of record	yr.	73†	74

* Includes additional identified pipe.

† Corrected figures.

Even the 76-year record of the Springfield system does not serve definitely to determine the past average lives of its cast-iron mains 6-inch and larger in diameter, as such a period is but a good beginning of what is indicated as the life of mains of these sizes. Nevertheless the data available in that system forms the basis for conclusions as

to the lives of pipe and the way in which retirements take place which, while tentative in the absence of records from more plants, are better founded than most past estimates. Furthermore, in view of the unusual combination of factors influencing retirement of mains in the history of the Springfield system and with present indications of decreasing population growths, better pipe material, the treatment of water and lining of mains to reduce corrosion, all acting to lessen future retirements, these conclusions will, in the opinion of the authors, prove to be conservative as compared with what will happen in the ordinary water system.

In extending the study of the Springfield experience, the additions and retirements of cast-iron pipe for the year 1939, as well as data for a portion of the previously unidentified pipe which a more intensive search of records disclosed, have been incorporated with the previous record. The resulting effects upon the mortality survival data have not changed appreciably the conclusions previously presented. Table 1 summarizes the data for 1,650,062 ft. of cast-iron pipe, compares it with the data previously reported, and includes percentages of survival now obtaining for the most recent ten years of retirement experience.

It has been recognized that mortality survival curves of units of physical property are generally in the form of the human mortality curve. The curve representing the 4-inch pipe experience and the beginning of the curves of experience with 6-inch and the larger sizes of cast-iron pipe in the Springfield system, as given in the October, 1939, JOURNAL, show that, except for the effect of infant mortality upon the human mortality curve, the curve for water mains will take the same general form. Furthermore, a comparison of the data for the various sizes of pipe and an analysis of the causes of such variations as appear to exist, as well as a study of other data to which the authors have had access, lead to the conclusion that, for all practical purposes, this 4-inch pipe curve can be converted to a percentage curve and be applied to the larger sizes of cast-iron pipe, with proper adjustment for the longer average service lives of such pipe.

By studying the effects of applying assumed variations in basic data upon the computations subsequently presented in this paper it has been found that no important changes will result in the conclusions reached because of such variations as may be expected when data covering the experience of a large number of systems becomes

available. Temporary departures of individual water plant experiences from the mortality curve, even though radical in nature, will in all probability have little final effect on a curve of general experience. Even if the curve itself ultimately changes its slope somewhat, the practicability of the method herein proposed for determining depreciation of cast-iron mains will not be affected and the basic percentages for use with various service lives will vary but slightly.

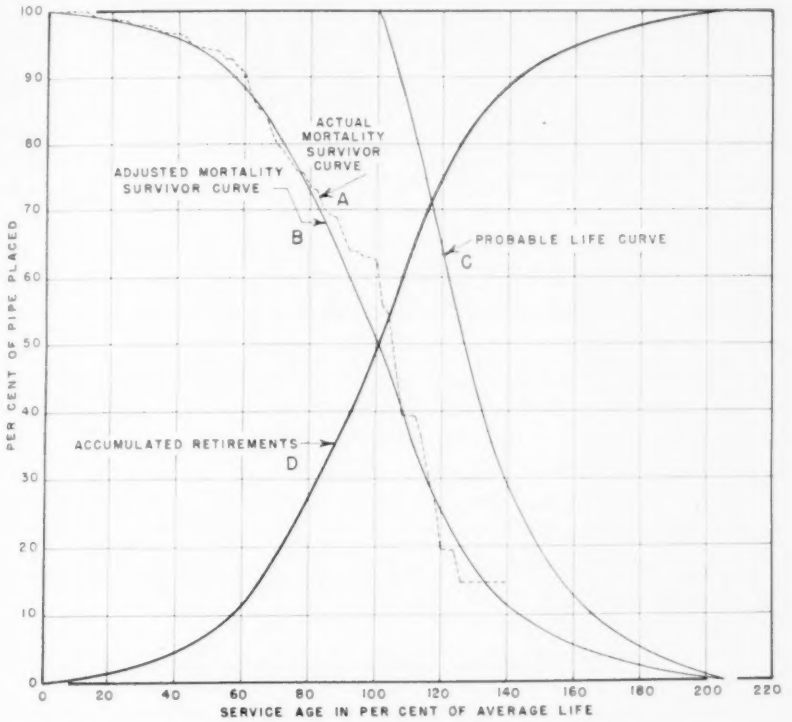

FIG. 1. Mortality Curves for Cast-Iron Pipe; based on pipe service records of Springfield, Mass.

Figure 1 shows the adjusted mortality Curve B for cast-iron pipe based upon the actual mortality data of Curve A. It plots the percentage of units surviving at various times in terms of percentage of the average life of the groups of units. Also shown is the corresponding probable life Curve C reflecting the remaining life expectancy of the units surviving at various times. The Curve D is the complement of Curve A and represents the accumulated retire-

ments, in percentage of the units installed, throughout the total life span of the last unit to be retired.

Accounting for Depreciation

The mortality data for cast-iron pipe show very plainly that retirements in such a group of items do not take place at a regular rate, that they occur mainly near the average life of the group and that they are very small during the early and late years of the life span. Since the rate payer must provide the amount of money necessary for the retirements and since the lives of mains extend over several generations it is important from his standpoint that consideration should be given to collecting this money as equitably as possible from the different rate payers who will use the property during its life span.

The rate of accumulation for these retirements is also of great importance to the owner of the property since in many cases these accumulations are given consideration in determining the value upon which he is allowed to earn a return. While for many years the retirement-reserve method of providing for retirements was approved in many quarters, the present tendency of the various regulatory bodies is to require some form of accounting for the depreciation in the remaining units of property. Where commissions have used a computation method of determining depreciation, they have, generally speaking, adopted either the straight-line method or some form of compound-interest method and it is with a comparison of these that this paper is primarily concerned.

Straight-Line and Compound-Interest Methods

It is assumed in the straight-line method that all factors causing the ultimate retirement of a unit of property are exerting their influence at a constant rate and that the loss of value takes place uniformly throughout its service life. The annual provision for depreciation is obtained by dividing the depreciable value by the estimated service life and the total required to retire a single unit of property is accumulated regularly during that period. If such accumulations are considered to represent the accruing of depreciation and the loss in value, it follows under this method that the value at any time is directly proportional to the age of the unit without consideration of any other factor.

The compound-interest methods have variously been termed the

sinking-fund method, the compound-interest or annuity method and the present-worth method. They are alike in that the accumulation of the depreciation reserve fund follows the growth of an annuity placed at compound interest. The term "sinking-fund method" has been rather loosely applied to all compound-interest methods whether or not a fund or its equivalent in property value was actually segregated. In most instances, in fact, no fund was established.

In a true sinking-fund method the value of the property on which a return is allowed continues at its original amount undepreciated. A consumer pays equal annual amounts as return on such property and equal annual amounts toward the reimbursement of the owner for the ultimate retirement of the property. These latter contributions by the consumer are placed in a fund by the owner; the interest earnings thereon are added to the fund and in turn earn interest in the future. This fund grows as a curved line, slowly at first and more rapidly as the end of the property's service life approaches, ultimately amounting to the original figure for the unit of property. Actually this procedure is rarely followed.

Instead of segregating a fund the more simple application of the compound-interest method is generally used. The total amount which the consumer pays each year toward providing for depreciation of a unit of property is made up of an annuity, which is the same each year, plus the interest on the total of the annuity and interest accumulations, as it existed at the beginning of the year. The total of these two amounts is charged as an operating expense but the consumer pays no more than if a fund were actually segregated because he pays a return on the depreciated value of the property instead of on the original figure. In other words, as time passes the amount the consumer contributes toward depreciation grows larger because the interest on the growing accumulated reserve is greater each succeeding year, but at the same time, if the reserve is considered to be a measure of the accruing depreciation, the amount he pays as a return on the value of the property decreases because the deduction of such a reserve from the original figure for the property leaves a progressively smaller amount as a base for computing the return.

The owner also gets a comparable total return under the sinking-fund or compound-interest methods. In the second procedure he receives less as return from the consumer but the money represented by the reserve, instead of being in a segregated fund, is available for him to invest either in extensions to his plant or elsewhere and the earnings thereon are his as compensation for the lessened return

received from the consumers being served by the original units of property.

It has been contended that, as a matter of equity to both consumer and owner of the property, the rate used in computing the interest element of the annual depreciation provision should be the same as the rate of return.

It is argued that the consumer, during the service life of a unit of property which he uses, receives the same service from it each year and should, therefore, pay the same amount annually as a return plus contribution towards its retirement. The amount of return he must pay is the product of the rate of return and the depreciated value of the unit serving him. The amount of the annuity, which is one element of his contribution toward depreciation, is the same each year. Therefore, if the sum of return, annuity and interest are to remain constant, the interest element of the contribution must be the same as the amount of return from which he has been relieved. Where the amount of the reserve which has accumulated is considered to represent the amount of the depreciation on which he is not required to pay a return, the interest on the reserve will equal the reduction in return only if the rate of interest is the same as the rate of return.

Further, it is pointed out that the owner of the property, who raised the capital in the first instance and took the risk in the enterprise, is entitled to continue to earn the fair rate of return on the whole amount of that capital so long as he continues to operate the utility and be responsible for the adequacy of its service. The reserve, if it is considered to represent the accumulated depreciation on the property under consideration, equals the amount on which he no longer receives a return from the consumers using that property. He, of course, cannot pay out the cash equivalent of the reserve in dividends as it will be needed sometime to take care of retirements. Until that time comes, however, he can invest it and the returns from it are his. From every point of view including the question of risk he is better off to put it into improvements to the plant. If this is done, while there may be some lag in many cases, in the long run he will, under regulation, be earning the allowed rate of return on the reinvestment. Thus he will receive in total an amount equal to the rate of return applied to the whole amount originally involved in the property under discussion.

In connection with such an assumption as to interest rate, the

relation between the present-worth and the compound-interest methods is noted. The present worth of a sum of money due at a future date is that amount of money which, if placed at compound-interest for the intervening period, will equal the amount due at the future date. Under the present-worth concept of depreciation, the value of a utility property at any time is equal to the summation of the present worths of each of the future yearly returns and contributions toward depreciation which it is capable of producing for the remaining years of its service life. At a time, x years after installation, the value would be the summation of the present worths of future return and depreciation provisions for a number of years corresponding to the service life less x years. The depreciation at the time, x years after installation, would be the difference between the then present worth of its future prospects and its value when new. It is obvious, therefore, that with the same period and rate of interest the compound-interest and present-worth methods produce identical results. Thus when the compound-interest method of computing depreciation is based upon the use of the same rate of interest as the rate of return, the total of the depreciation accumulations is the same at all points in the life span as when computed by the present-worth method.

While this approach to the problem is from the viewpoint of the owner in the first instance, in the case of a utility where the rate of return is prescribed by a regulatory body, it is contended that the application is equally sound from the consumers' point of view. What the owner is entitled to receive in the future is what it is proper for the consumer to pay for the service; and the worth today of such future receipts when evaluated at the allowable rate of return is the value of what the one has to sell and what the other plans to purchase. As property ages, due to the approach of the date of prospective retirement, the present worth of its future earnings decreases as its remaining active years become fewer. Under this concept, therefore, the difference between such values at different times is the depreciation that has taken place in the meantime.

Having in mind the considerable logic in the arguments in favor of using the same rate of interest as the rate of return in applying the compound-interest method, the curves that have been developed for comparison with those reflecting the mortality data are based upon an interest rate of 6 per cent which has frequently been allowed as the rate of return on water works properties in recent years.

Comparison of Mortality, Straight-Line and Interest Curves

In Fig. 2, Curves E, F, G, and H, when compared with Curve D, illustrate how far the straight-line method departs from the requirements to care for retirements of cast-iron pipe and how closely depreciation reserves, based upon the compound-interest method as above defined, conform to the accumulated-retirement curve. Curves

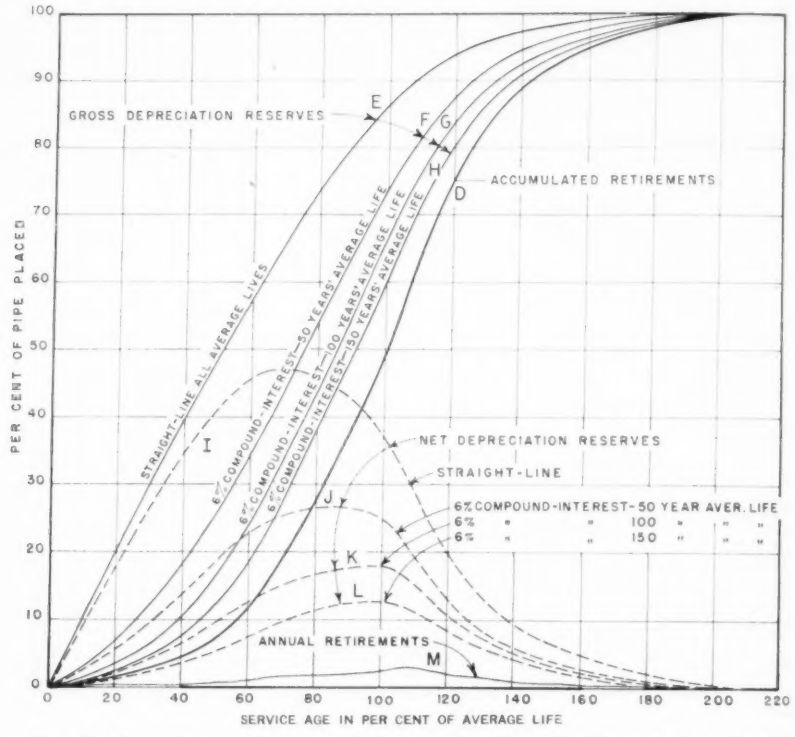

FIG. 2. Depreciation Reserves, Gross and Net, for Cast-Iron Pipe; based on pipe service records of Springfield, Mass.

D and E are percentage curves applicable to pipe of any size and service life. Curves F, G, and H are applicable respectively to 50-year, 100-year and 150-year average life property. They differ from each other because of the operation of compound interest for different periods of time.

It is seen that while the interest curves conform closely to the shape of the retirement curve and therefore to the need of reserves to cover

retirements, the accumulations are substantially in excess of such needs and provide for a material amount of depreciation in units still in service. Furthermore, the longer the service life involved, the nearer the depreciation-reserve curve approaches the retirement curve, which is logical from the viewpoint of the need for reserves to provide against the likelihood of variations from the average.

Curves I, J, K and L, corresponding respectively to Curves E, F, G and H, show the net amounts remaining in the reserves after retirements have been deducted under the straight-line method and for the three average service lives under the compound-interest method. Curve M shows, also in percentage, the yearly retirements. All values on these curves are expressed in percentages of the amount of pipe installed in a single year. The time shown is in percentage of the respective average lives.

From an examination of the curves in Fig. 2 the basis of certain conclusions of those who have advocated the use of a compound-interest method of computing depreciation rather than the straight-line method is apparent. The wide spread between Curve E, representing the accumulation under the straight-line method, and Curve D, representing the accumulated retirements, indicates that beginning in the early years, the depreciation reserve promptly attains a sum many times that needed for current retirements. Only in the last fifth or sixth of the service life of the longest survivors does the difference between the two curves begin to represent a reasonable margin above retirement needs and prior thereto it is materially out of proportion. Comparison of Curves I and M shows that at 20 per cent of the average service life the straight-line reserve is about 90 times the yearly retirements. At the 40 per cent point it is still over 85 times the annual rate of retirement and at the 50 per cent point, nearly 60 times. Not until the average age is reached does the reserve become as small as 15 times the annual retirements at which time the reserve, as later shown on Fig. 3, amounts to about 70 per cent of the property remaining in service.

As a result of straight-line contributions toward depreciation in such large amounts in the early years combined with the payment of a return on nearly new property, the consumers in the early history of the property make large total combined payments for return and depreciation. As time goes on, due to equal contributions to depreciation and diminishing amounts for return, the total of the two becomes much smaller.

TABLE 2

Comparison of Return and Depreciation—Pipe Group Value: $100,000

	RETIREMENT METHOD	STRAIGHT-LINE METHOD	6% COMPOUND-INTEREST METHOD
Age 0			
Property in service	$100,000	$100,000	$100,000
Value of property in service	100,000	100,000	100,000
Annual depreciation	100	1,981	578
Return at 6%	6,000	6,000	6,000
Total paid by consumer	6,100	7,981	6,578
Earnings on invested reserve at 6%	0	0	0
Total received by owner	6,100	7,981	6,578
Age 30			
Property in service	$89,600	$89,600	$89,600
Value of property in service	89,600	44,370	67,300
Annual depreciation	1,100	1,775	1,856
Return at 6%	5,376	2,662	4,038
Total paid by consumer	6,476	4,437	5,894
Earnings on invested reserve at 6%	0	2,713	1,338
Total received by owner	6,476	7,150	7,232
Total-To-Date, End of Year			
Annual depreciation allowances	11,500	57,404	34,551
Return at 6%	173,868	129,192	156,066
Total paid by consumer	185,368	186,596	190,617
Earnings on invested reserve at 6%	0	44,659	17,798
Total received by owner	185,368	231,255	208,415
Age 60			
Property in service	$27,200	$27,200	$27,200
Value of property in service	27,200	6,710	13,000
Annual depreciation allowances	2,200	539	1,009
Return at 6%	1,632	403	780
Total paid by consumers	3,832	942	1,789
Earnings on invested reserve at 6%	0	1,130	852
Total received by owner	3,832	2,072	2,641
Total-To-Date, End of Year			
Annual depreciation	75,000	93,830	88,020
Return at 6%	284,124	168,441	223,476
Total paid by consumer	359,124	262,271	311,496
Earnings on invested reserve at 6%	0	114,036	60,640
Total received by owner	359,124	376,307	372,136
Total of Group			
Annual depreciation allowances	$100,000	$100,000	$100,000
Return at 6%	302,800	172,600	232,100
Total paid by consumer	402,800	272,600	332,100
Earnings on invested reserve at 6%	0	127,400	70,800
Total received by owner	402,800	400,000	402,900

Table 2 represents a tabulation and summation of return and depreciation under the retirement, straight-line and 6 per cent compound-interest methods of providing for depreciation for a group of pipe representing a depreciable original amount of $100,000, whose average service life is 50 years and whose mortality survival is as shown on Curve B, Fig. 1. The annual amounts required for depreciation and return at 6 per cent and the earnings on the reserve fund are shown at ages 0, 30 and 60 years and the accumulated amounts at the end of 30, 60 and 103 years. Under the retirement method no reserve accumulates as the annual depreciation is considered to be the amount of the actual retirements.

It will be noted that in the early period in the life of the group the consumers pay considerably more for their service under the straight-line method than they do under the compound-interest method. In the first year this is $7,981 as contrasted with $6,578, or about 20 per cent more. It is not until the second decade that the payments become equal. Assuming an average of $67.00 of main pipe value per consumer, 1,500 consumers will pay at the beginning $5.30 per year return and depreciation under the straight-line method, and $4.40 per year under the compound-interest method. At age 30, based upon the assumption that as mains are retired a corresponding number of consumers are eliminated from further payments upon this group of pipe, 1,340 consumers will pay $3.30 per year under the straight-line method while the payment per consumer under the compound-interest method will remain the same, $4.40 per year. At age 60 the respective payments for the then connected 400 consumers are $2.35 and $4.40 per year.

It is apparent that under the straight-line method consumers pay more for the same service in the early period than they do near the end of the property's service life. Inasmuch as the situation involves property much of which continues in service for several generations, it is evident that different groups of consumers are paying differently for the same service. Those taking service in the early years are being discriminated against in order that others in succeeding generations may get the same service for less.

It will further be noted from Table 2 that the total received by the owner during the life of the longest surviving unit of property is approximately the same, $400,000, under the three methods. This is equivalent to a return at 6 per cent on the average investment during the total life span and a payment equal to the original investment to offset its retirement. Of this amount the consumers would pay

but $272,600 under the straight-line method, whereas they would pay $332,100 under the compound-interest method, and the entire amount under the retirement method. Under the straight-line method approximately 42.5 per cent of the owner's return must be secured by investing his money a second time while under the compound-interest method the proportion is 23.5 per cent.

With straight-line depreciation the owner of the property has a very substantial part of the money he put into the plant returned to him many decades before actual depreciation justifies it. He must become the trustee responsible for the investment of large sums of money, and as the community served begins to follow the trend of present-day population growth, such sums may easily be larger than he has need for in the plant itself. In the meantime he must continue to operate property very slightly depreciated in fact, but upon which he cannot earn a return because, due to an arbitrary assumption as to the way in which depreciation in the property is accruing and as to its amount, large deductions are made in the early part of its service life from the amount upon which he is allowed to earn a return.

The most striking effect of the arbitrary assumption made in straight-line depreciation accounting (that property depreciates regularly with the passage of time) occurs when a valuation is made some time after the installation of a group of property units and this method of computing depreciation is applied for the first time to determine the reserve requirement. In these circumstances the results of its application are particularly stringent in the effect upon the owner. For instance, referring to Table 2, if a valuation were made at age 30, the straight-line method determines the value of the $89,600 of pipe then actually in service to be $44,370. It assumes the consumer has paid in the 30 years a total of $186,596, or 68.5 per cent of the total return and depreciation which must be paid during the 103 years of life of the group. Where such methods of providing for depreciation had not been the practice in the past such a determination of value obviously deprives the owner of much of his property without hope of payment for it or the opportunity to recoup in future earnings. The property in service, amounting to 89.6 per cent of the original group, has been depreciated over 50 per cent in somewhat less than $\frac{1}{3}$ of the life of some of the units of the group, all without any basis of fact to indicate the need for such a deduction.

The above example is indicative of the effect of straight-line depreciation accounting in a group of properties having an average life of 50 years. As large as these reductions in value are they become much greater in property having materially longer lives. It is apparent that the straight-line method of accounting for depreciation should not be applied to the long-lived property in water plants simply because it has been adopted in many cases for electric, telephone and similar short-lived utility property. Its general application to water systems is not equitable either to the rate payer or to the owner because it is discriminatory as between consumers and confiscatory as to the property of the owner.

A study of the curves in Fig. 2 and the figures in Table 2 resulting from the use of the compound-interest method of determining depreciation reserves reveals that by the application of that method these errors and inconsistencies, inherent in the straight-line method, are eliminated or greatly reduced. Consumers in the early years of the life of a group of pipe will not be required to pay more than their share as return and contribution toward the retirement of the pipe. Instead consumers at all periods during the service life of the pipe group pay the same annual amount as return and depreciation for the same service. The total amount paid by consumers during the life of a group of pipe, in view of the service received, is a reasonable proportion of the total return and depreciation to which the owner is entitled.

The margin between the reserve and the retirement needs, though substantial, does not reach such unreasonable figures as it does under the straight-line method. The 100-year life reserve, for instance, reaches a maximum at 40 years of 16 times the current retirement rate and at 100 years is 7 times the requirements. The amount of reserve built up, while effective as protection against variation from the average rate of retirements, is within the limits of what can reasonably be reinvested in the property. The owner, therefore, can probably get from the earnings of the reserve about the amount of return he no longer receives from consumers using the part of the original group of property still in service without having to go outside the field of business with which he is familiar.

If the property is valued at any time, either for rate purposes or for sale, the owner is not penalized by the deduction of a large theoretical depreciation in no way related to the way in which reserves will be

required to cover retirements. The compound-interest curves show a consistent relation between the growth of the depreciation reserve and the accumulation of retirements and there is an element of reserve above retirement needs applicable to depreciation in the units of property remaining in service.

The assertion is often made that, whereas straight-line accounting is simple, compound-interest accounting is too complex to be usable and further, that the compound-interest method cannot readily take account of the fact that in a group of units having a given average life some of such units are retired before the group attains its average age and that because of the nature of compound interest the total amount required for all retirements will not be accumulated.

Neither of these assertions is a valid objection to the compound-interest method. Within the degree of accuracy that is justified by the assumptions that must be made in applying either form of depreciation accounting, it is not difficult to compute accrued and annual depreciation by the compound-interest method as is demonstrated subsequently in this discussion.

In the first place the problem concerns itself with very long-lived property and the average service life to be used in the accounting must of necessity be an estimate. Therefore, mathematically accurate computations of the accounting entries to be made would be inconsistent and uncalled for. In the second place this is a comparison of two methods of estimating depreciation, one of which produces results which at certain periods are five times as great as the other so that the question of difficulties in obtaining extreme accuracy in computations is not pertinent. Furthermore, the weight to be given to the assertion that a method of depreciation accounting requires laborious accuracy must be decided in the light of the fact that it comes from advocates of another method that produces a reserve which, during much of the life of the property, is from 50 to 90 times larger than the probable annual retirements. In the circumstances as they exist the application of the compound-interest method as illustrated in this paper meets the justified requirements of accuracy. Proper accounting, to reflect the retirements shown by the mortality curve, is not difficult; and, in view of the great distortion of values resulting from using the straight-line method as an alternate because it is simple, if depreciation is being determined by computation, the compound-interest method is well worth the small amount of work involved in its application.

Group Accounting for Depreciation

While the basis of accounting for depreciation of single units or for groups of units is theoretically the same, actually in carrying out such accounting some points of difference are encountered. Of a group of items having a particular average life, some items will be retired before, others after, the average age. Generally the survival of the units composing such groups conform to a mortality curve similar to that for cast-iron pipe. In such cases the depreciation of the whole group includes both actual retirements throughout the life span of the group and some loss in value in the remaining items by reason of prospective retirement.

In accounting for the whole group, therefore, account must be taken of the fact that accumulated depreciation allowances for the items of the group retired early are not sufficient to accumulate their full value and that these deficiencies must be made up in the accumulations on the items having lives longer than the average.

Where the straight-line method is used the percentage applicable to the property in service must be continued throughout the lives of all units of the group to accumulate a reserve sufficient to provide fully for the retirement of the group. A similar procedure is applicable to any compound-interest method. Further, in the latter, because a portion of the annual provision for depreciation is an interest accumulation and the reserve upon which such interest item is based is continuously being reduced by charge-offs due to retirements, the percentage rate applied to determine the annuity must be adjusted.

This adjustment is made by substituting for the percentage determined by the straight application of the compound-interest method a percentage representing the ratio of the present worths of the future retirements to the present worths of the property in service. The computation requires the summarization of the present worths of the retirements to be made year by year according to the mortality curve and a similar summarization for the pipe remaining in service each year. Because the present worths of the retirements made prior to the average age are greater than of those made afterwards, the percentage thus determined will always be larger than that for the compound-interest method based on all retirements taking place at the average age. This percentage, once determined for pipe of any selected average service life, remains the same and is applied annually to the value of all such pipe in place to determine the annuity to be

added to the reserve. Interest upon the reserve at whatever basic rate is being used is added and retirements are deducted as in the unit basis of accounting.

In the application of the compound-interest method based on 6 per cent, the percentages to be used to determine the annuities for unit accounting unadjusted and as adjusted for group accounting in accordance with mortality Curve B for the different average lives are as follows: for 50-year, the unadjusted is 0.3444 per cent and the adjusted 0.5781; for 100-year, unadjusted, 0.0177 per cent and adjusted, 0.1285; and for 150-year, unadjusted, 0.00096 per cent and adjusted, 0.0590. The curves shown in Fig. 2 reflect the adjusted percentages required to allow properly for retirements in accordance with the mortality curve of cast-iron pipe in Fig. 1.

Depreciation Reserves in Growing Plants

Curves D, I and J in Fig. 2 are brought forward to Fig. 3. In this latter figure ordinates are again in percentage but the abscissas are in years because this figure contains specific data pertinent to cast-iron pipe of 50-year average service life.

Curve N is the ratio of the amount of the straight-line reserve to the amount representing the units of a single group of pipe remaining in service, as contrasted with Curve I which is the ratio to the total of the group when placed. Curve P is the corresponding ratio-to-pipe-in-service curve for the 6 per cent compound-interest reserve for 50-year average life pipe. It is a companion curve to Curve J based on total pipe placed. Curves R and S show respectively what happens to Curves I and J when a similar sized group of pipe is installed each year and retirements take place out of all such groups in accordance with the mortality curve used as the basis of this discussion. Such installations and retirements result in the net pipe growth indicated by Curve W. Under this assumption the pipe installed just equals the retirements and the plant is static after 100 years and in fact is substantially so after about 65 years.

Curves T and U represent the ratios of these reserves, in plants growing as indicated by Curve W, to the amount of pipe in service. It will be noted that when the growth of 50-year average life pipe has become static, the reserve, according to the straight-line method, levels off at approximately 44 per cent of the pipe in service. The 6 per cent compound-interest reserve for pipe of this service life levels off at 23 per cent, approximately half as much. A study of

the relationship of the high points of Curves I (47.5 per cent) and J (26.8 per cent) with the corresponding leveling off points of similar growing plant Curves T (44 per cent) and U (23 per cent) indicates what may be expected of 100-year and 150-year pipe reserves in growing plants. The former would apparently level off, when growth became static, at about 14 to 15 per cent and the latter at between 9 and 10 per cent.

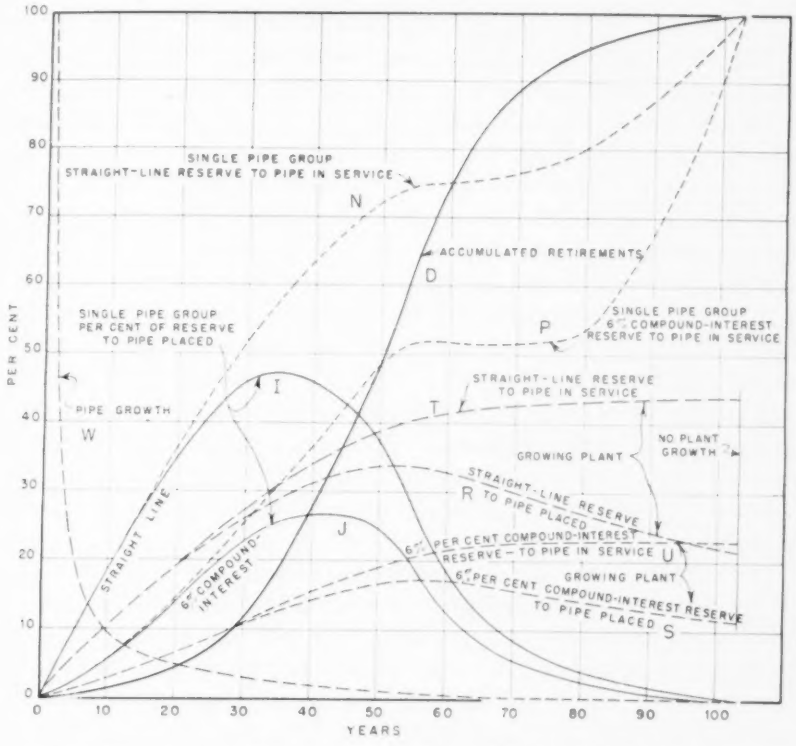

FIG. 3. Depreciation Reserves for Cast-Iron Pipe with 50-Year Average Life; based on pipe service records of Springfield, Mass.

It should be noted that as long as a plant continues to install larger quantities of pipe than is being retired, its reserves will not reach the leveling off percentages because of the effect of new pipe. Bearing in mind that few water plants have pipe of an average age of 30 years and that most of them are below 20 years, the percentage of reserve required in a growing plant at that period is of particular interest. Curve U in Fig. 3, representing pipe growth as shown by

Curve W, indicates that the reserve requirement for pipe, with average ages between 20 and 30 years and an average service life of 50 years, is from 6.5 to 11 per cent of the undepreciated value of the pipe in service. Similar curves for 100-year and 150-year average life pipe would obviously show lower percentages of reserve requirements.

Methods of Computing Accrued and Annual Depreciation Based upon Mortality Survival Data of Cast-Iron Pipe

The method developed herein for determining depreciation of cast-iron pipe by computation will be illustrated by reference to curves corresponding to pipe with a 50-year average service life. In making the determination of the reserve requirement it is necessary to use percentages as represented by a curve such as P in Fig. 3. Such a curve, once developed for the applicable mortality data will serve for all pipe of the same average-life group.

This curve reflects the following considerations: (1) the use of the compound-interest method of accumulating a reserve; (2) the use therewith of a rate of interest of 6 per cent; (3) an adjustment of the annuity percentage to take account of retirements before the average age in accordance with applicable mortality data; (4) the deduction, from the gross reserve that would thus accumulate, of the retirements as indicated by that data; and (5) the ratio of the net reserve so determined to the amount of the original group of pipe remaining in service.

To determine the depreciation reserve requirements for cast-iron water mains the percentage corresponding to the age of the pipe as shown on a curve developed as was Curve P is applied to the undepreciated value, on whatever basis is being used, of the pipe in service which was installed in any one year. The resulting figure is the reserve requirement, as of the date of valuation, of all the pipe of that group installed in that one year.

The summarization of such figures for each of the years in which pipe of similar life in the current inventory was installed is the reserve requirement for all such pipe. By adding the summaries for pipe of the various service-life classes, the total of the reserve requirement for all water mains in a system is obtained.

The annuity element of the annual provision for depreciation in pipe of 50-year average service life corresponding to a 6 per cent compound interest rate is determined by applying to the unde-

preciated value of all such pipe in service, irrespective of age, the factor of 0.58 per cent as indicated above. This factor does not change and the annuity element can subsequently be figured for each year by applying it to the amount of such pipe in service at the beginning of the year.

The annuity element for pipe of other average service lives is determined by applying similarly the respective factors to the total value of each class in service and by totaling all of the resulting figures the total annuity element is found.

The interest element of the annual depreciation provision is determined by a single application of 6 per cent to the amount of the reserve as determined above. After this system of accounting is in operation the percentage is applied to the amount of the reserve at the beginning of each year.

There is thus determined the entries required for establishing and keeping in operation a system of accounting for depreciation of water mains by computation which is closely related to the way in which the evidence at hand indicates actual depreciation to take place.

Conclusions

From the results of the study of mortality data of cast-iron pipe determined from the records of the Springfield system, and drawing upon the experience of the authors in the operation and management of similar water works systems, the conclusions reached by them with reference to accounting for depreciation of cast-iron pipe systems may be briefly summarized as follows:

(1) Whether plants be privately or publicly owned, the same principles of correct application of depreciation accounting should be observed.

(2) Cast-iron pipe systems of water utilities have, on the average, substantially longer lives in service than has hitherto been considered to be the case. Four-inch pipe may be expected to have an average service life of 50 years or more, the service life of 6-inch pipe will average materially over 100 years and that of larger pipe will be 150 years or more. These expectancies of life should be revised upwards in those plants where conditions of service and growth are relatively satisfactory and stable. In particular cases, due to certain types of local conditions, shorter life expectancies may be indicated. Final estimates of lives should be adopted only after careful study of the factors influencing the retirement of pipe in each plant or divisions of plants.

(3) In the light of present knowledge as to the rate at which retirements take place, the mortality and probable life curves, B and C in Fig. 1, may reasonably be assumed to reflect generally the rate of retirements for cast-iron pipe and the probable expectancies of surviving units.

(4) The methods presently used to determine depreciation vary widely in the results obtained and, in many cases, are incorrectly applied and bear little or no relationship to the actual depreciation that has taken or is taking place in cast-iron pipe systems.

(5) The straight-line method of accounting for depreciation has no basis of fact or reason, other than simplicity, to dictate its use; particularly as it affects accounting for depreciation in long-lived property, such as water mains, it is arbitrary, destructive of value, exorbitant in its demands upon the early rate payers and bears no relationship to the way in which water mains are retired or depreciate in value.

(6) The compound-interest method, using an interest rate approximating the rate of return, appears to fullfill the purpose of relating a method of computation for determining depreciation in cast-iron mains to the way retirements may be expected, with sufficient reserve to allow for a considerable variation from the basic data used and yet having a substantial provision for accrued depreciation in units of property remaining in service.

(7) The compound-interest method, correctly applied, is little more difficult to apply than those based upon straight-line or other arbitrary assumptions but the results of its application are so much more equitable to all concerned, particularly in long-lived elements of property, that, even if slightly more difficult to apply, its use is warranted where depreciation is to be determined by computation.

(8) The reserves built up, and currently being built up, by the straight-line and certain other methods of accounting for depreciation have become and are becoming excessive and the continued use of these methods will result in the management of water works properties becoming primarily problems of investment with materially less time available for those of effective operation.

(9) Water consumers, in the long run, will not benefit from a method of depreciation accounting which is not just as fair to the owners of the plant which furnishes water service to their community as it is to the rate-payers.

Lining Cast-Iron Mains in Position With Cement Mortar

By J. E. Gibson

CEMENT-MORTAR lined and encased sheet-iron and steel-riveted water mains, and cement-mortar lined wrought-iron and steel service pipes have been in use in the United States, particularly in New England, for almost a hundred years and are still giving good service. It was not until 1922, however, that the practice of cement-mortar lining of cast-iron water mains was introduced. The use of cement-lined and encased sheet-iron water mains was more than likely dictated by the high cost of the cast-iron bell and spigot pipe of that time. The increasing demand and improvement in the manufacture, perfected by the pipe foundries, soon overcame the price differences in the two materials, so that by the late nineties the manufacture of cement-lined sheet-steel pipe had almost ceased. The last company making this type of pipe was the American Pipe and Construction Company of Philadelphia, with which company the author was associated as assistant engineer to the late J. W. Ledoux, Chief Engineer.

Experience gained and flow tests made on cement-lined wrought-iron pipe in comparison with tests on cast-iron mains of the same age, laid in the same system and carrying the same water, showed the superiority of the cement-lined over the cast-iron mains. Cast-iron mains, 20 years of age, on test gave values of C in Williams and Hazen formula as low as 40 or 50; whereas the coefficient obtained in similar tests made on the cement-mortar lined pipe of like age in the same distribution system remained constant, and in some cases improved, demonstrating conclusively the hydraulic advantages of the cement-mortar lined pipe. It was upon the experience and

A paper presented on April 25, 1940, at the Kansas City Convention by J. E. Gibson, Manager and Engineer, Water Dept., Charleston, S. C.

information gained from these tests that the author proposed to the Commissioners of Public Works of the City of Charleston, S. C., in 1921, to line standard cast-iron water mains with cement mortar. Favorable consideration of the suggestion was received from pipe foundries, and in June, 1922, in cooperation with the American Cast Iron Pipe Company of Birmingham, Alabama, experimentation with the lining of standard cast-iron pipe was initiated. During the fall of 1922, the Charleston water department laid some 10 miles of cement-lined cast-iron pipe from 4 in. to 24 in. in diameter (1). Tests made immediately after laying this pipe in 1922, to determine the value of C (Williams and Hazen formula) gave values as high as 140 and averaged 135; and further tests within the last 18 months show that these values have been maintained, and in a few instances have improved. Coefficients of 140 may be conservatively used for cement-mortar lined mains of cast-iron construction, even when based on the nominal diameter of the pipe when the lining does not exceed $\frac{1}{8}$ in. in thickness.

The cleaning of cast-iron water mains that have become tuberculated has been practiced for the last 30 or 40 years, but experience gained has shown that, with many soft active waters, cleaning is of doubtful value, as the process of scraping the mains removes the inhibited rust coating formed by the tubercles, and further destroys the tar-coating, thus offering fresh unprotected surface and accelerating the formation of new tubercles. This rapid growth of tubercles increases the interior roughness so that cleaning becomes an annually recurring expense and an annoyance to the water consumers.

The executives of the Charleston water department, faced with their disappointing results from the cleaning of mains, and with their excellent experience with the cement-mortar lined mains, began to look for some practical method of protection of the exposed surface of the water main after cleaning.

In about 1930, W. T. Tate of Sydney, Australia, began experiments with a process to cement-mortar line cast-iron mains in position, and in March, 1932, was granted Australian patents covering his process. He began the actual lining of mains in the Sydney Metropolitan Water Supply and Sewerage District. At the close of 1935 approximately 89 mi. of mains had been lined in position and by the end of the year 1938, over 600 mi. had been completed; and to date, over 1,000 mi. In 1934, Mr. Tate demonstrated his process of lining mains in position to the Water Works Commission of

Manchester, England, and it was a description of the work done at Manchester, as presented in a paper by Thomas H. Wiggin, Consulting Engineer of New York City, read at the convention of the American Water Works Association held at Cincinnati, Ohio, in 1935 (2), which first called the author's attention to the Tate process of lining. The Tate Pipe Linings, Inc., of Andover, Mass., was incorporated October 25, 1938, with E. H. Lewis of Andover as president. The Company obtained the license for the use of Mr. Tate's patents in the United States. Bruce Harkness of this company read a paper in February, 1939, before the New England Water Works Association (3), giving his experience in lining water mains under the Tate patents, particularly in England and in Buenos Aires, Argentina; and later he took up with us the proposition of lining mains in Charleston, S. C.

As a result of the information gained from the above papers and a study of the process and equipment as presented by Mr. Harkness, the water department of the City of Charleston entered into a contract in July, 1939, for the experimental lining of about $2\frac{1}{2}$ mi. of 4-, 6-, and 8-inch water mains in the system of the Charleston water department. The work done under this contract is described in this paper.

Description of Procedure, Process and Equipment

The procedure of cleaning and lining has been developed into a technique. First, the section of main to be cleaned is isolated from the rest of the system, and to take care of the consumers, if numerous, a small service main is laid along the gutter or curb in the street with service outlets provided for each house. This temporary service main is then connected to the water main at convenient locations, preferably back of valves in the section to be cleaned. If valves are not available then the main must be cut and plugged to take care of the feed to the temporary main without interference with the work. The street is then opened at all service connections and all services are disconnected. A section of hose is inserted between the service pipe and the temporary main at the gutter, so that during the interval of cleaning and lining the consumers receive water through the temporary main and the flexible hose connection attached to their regular service lines. When there are only a few consumers on the main being lined, water can be supplied them through the use of large containers delivered by truck. Next, all

hydrant leads, tees, and cross-mains are disconnected from the section to be cleaned. The main at the ends of the section to be cleaned, usually about 300 to 400 ft. in length, is then cut so as to permit the introduction of the cleaning apparatus. By means of sewer rods or steel tape a cable is introduced through the section of main to be cleaned. This cable is attached to a single drum hoist mounted on a truck, there being two trucks, one at each end, so the cable can be pulled back and forth alternately, each truck having a cable of sufficient length to pass through the section of main being lined.

The cleaning equipment includes scrapers as shown in Fig. 1. These consist of a heavy wrought-steel frame work upon which are

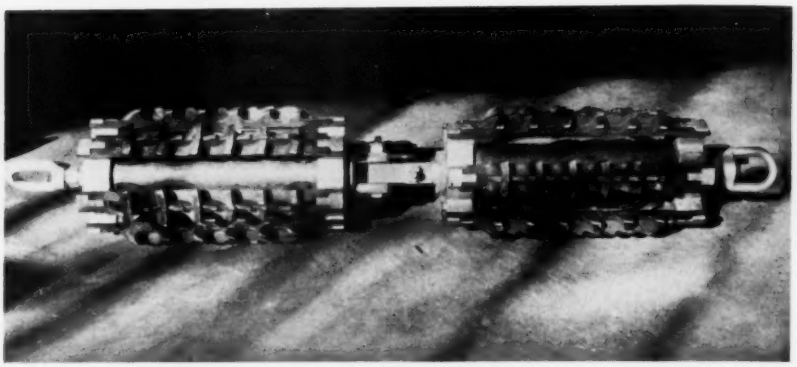

FIG. 1. Tate Cutting Scrapers. Note elliptical springs to hold cutter blades against wall of pipe

mounted cutter blades; the complete equipment consisting of two sections, the cutting blades in the two sections being staggered or rotated so that the trailing section cutter blades cover the circumference area of the pipe not scraped by the leading blades. These blades are made up of heavy cast-steel sections mounted on a spider and held out to position by flat elliptical springs. Each section has three cutting blades for 4-inch pipe, and six cutting blades for 6- and 8-inch pipe, and all cutting blades have six cutting edges or knives. Blades are spaced symmetrically around the circumference of the spider and the knives are arranged in increasing diameter so that there is in effect a series of cuts made on the incrustations or tubercles on the walls of the main. The knives of the cutters are designed to be self-sharpening. The cuttings fall down through the

spaces between the blades and with little exception most of this material is left in the main for a later removal. These scrapers are passed back and forth two or three times through the main, depending upon the amount of incrustation or tubercles to be cut from the walls.

Scrapers are of rugged construction and can readily remove any obstruction in the main such as overrun lead joints or protruding unchartered corporation stops, should such exist. (It might be well to note here that it is very important to have all corporation stops and services removed from the main in advance of the final lining operation, for if not they may cause the cement-lining to fall and form an obstruction in the main.)

After all adhering tubercles and foreign substance have been cut from the walls, there is then passed through the main a series of scraper brushes made up similar to the scraper section except that the knives are replaced by sections of steel brushes. These steel brushes built like the heavy ones used for cleaning steel plate for painting are held in a manner similar to that of the cutter section described above. Attached to the trailing end of the wire brushes are rubber squeegeeing pistons which are made up of steel plates with rubber discs inserted between the plates (Fig. 2). The squeegeeing sections have a double function; first, they trail the wire brushes and force out ahead of them all of the material that has been cut loose with the scrapers and the wire brushes, so that the main is thoroughly cleaned of all loose foreign material. The brushes and rubber squeegeeing pistons are passed through the section three or more times, after which the brushes are removed, and the same section or a new section of squeegees are passed through the main. This second operation is to remove all moisture, water, etc., that is lying in the main at low points, as the main must be entirely dry and free of all moisture. In Charleston it was found that after passing these squeegeeing pistons through the main, on the final exit the rubber was warm to the hand. When the latter operation is completed, all corporation openings, crosses, tees, and outlets must be plugged so that nothing protrudes into or through the walls of the main, but so that each opening is effectively closed. For fittings such as tees and crosses special plugs made slightly smaller in diameter than the bore of the pipe are inserted in the horn of the tee or cross and held in position by clamps. These plugs are made smaller

in diameter than the bore of the pipe so that the lining mortar may be forced out around the plug and line the horn of the outlet to the desired thickness of lining. After squeegeeing the main for moisture, and before lining, the proper practice is to draw a proving plug through the main to make certain that there is no obstruction in the main that would interfere with or injure the lining machine during the later operation. This proving plug is made of metal or wood that is slightly larger in diameter than the machine to be used in lining (Fig. 3). It is probably more desirable to pass the proving

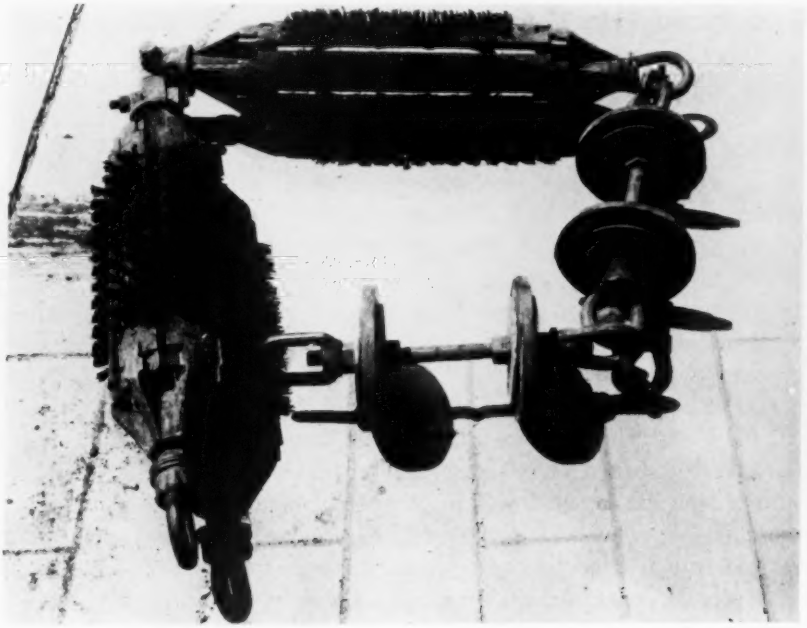

FIG. 2. Tate Wire Brushes and Squeegee Pistons Assembled

plug by using man power as the operator can thereby obtain a more sensitive survey of conditions in the main. After the proving plug has been passed through satisfactorily, the main is ready to be lined.

Toward the end of the squeegeeing operation, a second gang of men is set to mixing the mortar for the lining. This mortar is composed of one cement and two sand, and is mixed in a steel mixing pan by hand. The procedure is, as in mixing any other mortar, first, to spread the sand out in an even layer over the bottom of the pan, then add the proper quantity of cement. Square-nose shovels

or billies are used to mix the sand and cement thoroughly until an even color is obtained, after which the necessary water is added to give a satisfactory slump test (Fig. 4). It is important to have the mortar mixed to the proper consistency or slump and the amount of slump depends upon the size of the main being lined and the method used in introducing the mortar into the main. If introducing mechanically, a lower slump can be used than if introducing by hand. The Tate Company has, since its operations in Charleston and later at Falmouth, Mass., developed a mechanical device for loading the cement-mortar into the main (Fig. 5). This mechanical loader or charging device consists of a light, wrought-iron pipe con-

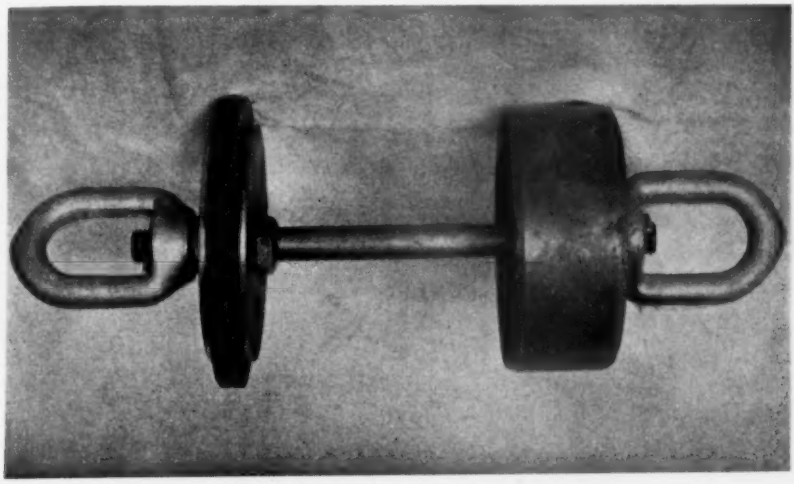

FIG. 3. Tate Metal Proving Plug

nected to the end of the main and a combination rubber and metal plunger attached to the cable used in cleaning. The cable is pulled up through the pipe so that the plunger is outside the charging device; the vertical section pipe is then filled with the mortar, after which the plunger is placed in the pipe and pulled downward by means of the cable and the hoisting machine until the mortar is pulled down into the main. The process is then reversed, the plunger pulled back, and a second charge of mortar introduced. This is repeated until a sufficient quantity of mortar has been introduced to line the section of pipe cleaned. This is the more desirable method of introducing the mortar, for as above stated, you can have

FIG. 4. Making the Slump Test before Introducing Mortar into Main—9½-inch slump for hand charging

FIG. 5. Mechanical Charging Device in Place Ready for Loading

a lower slump test and less semi-laitance to be squeezed out of the mortar in the lining operation.

After a sufficient quantity of mortar has been introduced into the main, the loading device is disconnected and the lining machine is attached to the cable. This lining machine (Fig. 6) is a conical or bullet shaped device having a hollow steel frustum of a cone attached to the down-stream end, all mounted on a steel shaft or spindle. This leading or bullet section of the lining machine has two sets of four heavy elliptical cantilever springs set at 90 degree angles to each other, and slightly larger in diameter than the main being lined. The function of the springs is to hold the lining machine in the center of the pipe so as to obtain a concentric lining in the main. The hollow skirt or conical section of the liner is perforated with a large number of conical openings bored from the inside, so that any material entering the openings from the outside—such as sand and cement—will free itself and not clog the openings. This is an important feature of the lining machine as it is this portion that performs the function of dehydrating the cement-mortar, allowing the entrained air, excess water and the semi-laitance to pass through and be deposited gently on the bottom of the new lining. The last $1\frac{1}{2}$ or 2 in. of the conical skirt of the lining machine is perfectly plain and smooth, and trowels out the mortar that has been plastered against the wall of the main by the pressure of the advancing head of the liner. As implied in the above, the function of the bullet and conical skirt of the lining machine is to force the mortar out against the walls of the main as the machine is being pulled through the main by the cable, the semi-laitance passing through the dehydrating openings and the smooth tail portion of the conical skirt acting as a trowel or float to smooth the interior surface of the mortar. It is important that the spindle upon which the bullet, springs, and perforated skirt-piece of the lining machine are mounted be straight and true in alignment.

There is always introduced into the main for lining an excess quantity of mortar, which is finally extruded at the opposite end. This excess mortar is required to line the horns of any tees and crosses, to fill any open spaces at the bell joints, and, as explained later, to act as a plunger in front of the lining machine, so that the cable is under tension and the lining machine under considerable pressure. Experiments made to determine the pressure immediately in advance of the skirt of the machine show that at times the pressure

there may exceed 150 lb. per sq.in., depending, of course, upon the slump of the mortar, the volume of mortar in advance of the machine, the size and angle of the cone of the lining machine and the grade of main being lined. Over the outlet end of the pipe there is placed a canvas sheet or bag which is tied to the end of the pipe and is held close against the cable used for pulling the lining machine. The purpose of this bag is to create pressure on the excess mortar as it is extruded through the opening in the bag, to assure a continuous and

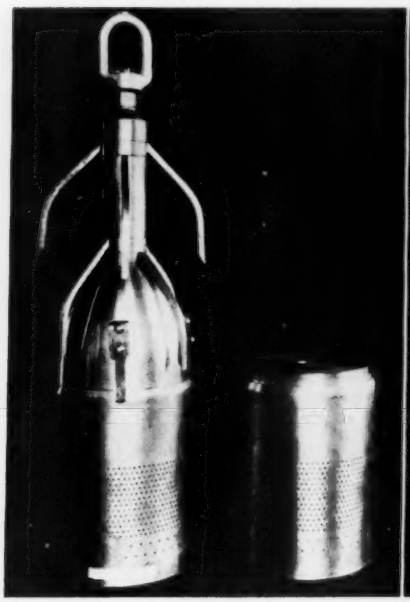

FIG. 6

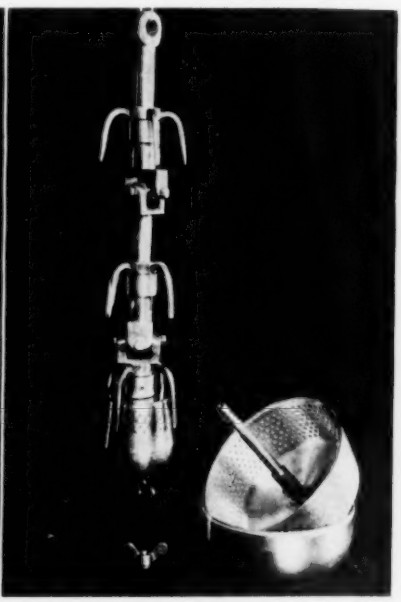

FIG. 7

FIG. 6. Tate Lining Machine and Extra Dehydrating Cone Piece
FIG. 7. Special Lining Machine for Elbow Offsets and Specials

uniform lining of mortar on the wall of the main. Care must be exercised to maintain the pulling cable in the center of the main as the liner approaches the exit end, otherwise the machine may be pulled out of the center and an eccentric lining result. It can readily be seen that when the lining is successfully completed the interior surface of the main presents a continuous smooth cement-mortar pipe without joints.

After the main is lined it is important to remove immediately the

plugs from the tees and crosses, so that they will not be held in position by the setting mortar that has been used in lining. At this time inspection of the interior of the pipe, by means of flashlight and mirrors, should be made, particularly at the larger size openings such as tees and crosses. Repairs can then be made to any damage to lining due to the removal of plugs from fittings. As soon as this inspection is made and everything is found satisfactory, all openings into the pipe, including the entrance and exit ends, should be covered with canvas bags to keep the air from circulating through the main, and to obtain the benefit of the heat of crystallization of the cement mortar in setting and of the even temperature of the soil, thereby obtaining ideal conditions for curing. Usually within 15 to 20 hours the main can be restored to service, previous to which time the plugs used to close the tap openings at corporation stops may be removed and the section of lining covering tap openings cut out with either a tool or a knife. In the work at Charleston a hand tool made from old combination drills and taps was made serviceable by sharpening the drill end and inserting a tee handle on the shank end. These were found very efficient and the men could readily drill the cement-mortar out, at the same time cleaning the thread in the tapped holes. The corporation stops may then be restored and the inlet and outlet entrances used for lining closed by inserting a short section of lined pipe, using a solid sleeve, after which the main is filled with water and allowed to cure a further period of from ten to twelve hours. After curing with water for several hours the main is ready to be restored to service by blowing off the curing water through hydrant connections or service line.

For lining bends, off-set pipes, and like fittings, the company has developed a special articulated short type lining machine (Fig. 7).

Important features in the process are: first, the thorough cleaning of the main and as heretofore stated the removal of all corporation stops and connections; and second, the quality of the sand and cement as well as the dehydrating orifices in the skirt of the lining machine.

The preparation in advance of lining and putting the line back in service require about 90 per cent of the total time, the actual lining requiring only a few minutes. Usually, depending upon conditions, about 800 ft. of main can be lined in a working day of 8 or 10 hr., but in the work at Charleston, which was experimental, progress was slower. The work was distributed all over the city in an effort to get a comprehensive picture covering all conditions from ordinary dirt

streets with few or no services, to congested residential sections where services averaged one every 25 ft., and under modern improved pavement, such as asphalt surface on concrete base.

The quality of the sand and cement used is very important, but of course it is desirable always, if possible, to purchase material that is obtainable locally in the market, and this was the procedure at Charleston. The cement used should have a reasonably long initial setting time. Pennsylvania Dixie Cement Corporation cement, having the following characteristics, was used at Charleston:

Chemical Analysis:		
Silica	22.10	
Alumina	5.61	
Iron oxide	2.29	
Lime	65.12	
Magnesia	1.33	
Sulfur trioxide	1.50	
Loss on ignition	1.86	
		99.81
Insoluble residue		0.35
Compounds:		
C_4AF	6.97	
C_3A	10.97	
C_2S	21.16	
C_3S	54.62	

Physical Properties:

Fineness ..... 93½ per cent through a No. 200 Sieve

Specific surface ..... 1750 sq.cm. per gram

Setting time, with Gilmore needle, initial 3 hr.; final 5 hr.

Tensile strength: proportion 1 to 3 standard Ottawa sand briquettes:

250 lb.—3 days
340 lb.—7 days
430 lb.—28 days

Cement was manufactured in every respect to meet the standard specifications for portland cement, A.S.T.M., and Federal Specifications SS-C-191a.

The sand used was a sand obtained locally and known as the Salkehatchie pit, located near Yemassee, South Carolina. The grading test on this sand showed the following:

Retained	14	mesh	0.49 per cent
"	28	"	10.58 per cent
"	48	"	67.15 per cent
"	100	"	17.58 per cent
Passing	100	"	4.23 per cent

The sand as received had the normal pit moisture and it was necessary to dry and screen it. Because proper screens could not be obtained, No. 16 mesh galvanized window screening was used and all material failing to pass this mesh screen was rejected. The sand, after it was dried and screened, was sacked, 1 cu.ft. to a sack, and hauled from the storage bin to the job. The proportions of mortar used in lining were one to two and the slump test averaged from $7\frac{1}{2}$ to 9 in., depending upon whether hand or mechanical means were used in charging the mortar into the main. The results of a test of the mortar as mixed on one of the jobs are shown in Table 1. In this particular case the slump was 10 in., a little bit in excess of what was normally used for hand charging.

In the experimental work, the aim was to place approximately $\frac{1}{4}$ in. of lining in the main, but it was not at all times possible to

TABLE 1

Results of Mortar Test

Tensile Strength

TIME	AS PLACED IN THE PIPE	SURPLUS CEMENT MORTAR AS EXTRUDED FROM THE MAIN LINED
	lb. per sq.in.	*lb. per sq.in.*
1 Day	175	180
3 Days	317	318
7 Days	417	467
28 Days	580	510

Curing for the first 24 hours in moist air, remainder of the time in water.

maintain this thickness as a number of sections of pipe varied as much $\frac{1}{8}$ in. over to $\frac{1}{8}$ in. under the nominal diameter. It was necessary to line one 8-inch section with a lining machine not exceeding $7\frac{1}{2}$ in. in diameter, giving a $\frac{3}{16}$-inch minimum and a $\frac{5}{16}$-inch maximum lining; but generally a thickness of lining close to $\frac{1}{4}$-inch was obtained. This is probably somewhat thicker than necessary, for it is believed a $\frac{3}{16}$-inch lining is just as good and probably as easy to obtain, provided, of course, there is a regular bore in the main. The quantity of cement-mortar needed varied with the diameter and averaged as follows for a section of 350 ft.: 4-inch main, 105 ft., 6-inch main, 80 ft., 8-inch main, 60 ft. In all cases this quantity of mortar gave sufficient mortar to line the pipe completely, together with any connecting branches for hydrants, street crosses, and tees, and an ample surplus extruded from the outlet end of the main.

TABLE 2

Tabulation of Information and Results of Lining 4-, 6-, and 8-inch Cast-Iron Mains in Position in Charleston, S. C., May to August, 1939

LOCATION	NOMINAL DIAMETER	FLOW CU.FT. PER MIN.	1000 GAL. PER 24 HR.	LOSS OF HEAD PER 1000 FT. OF PIPE	AVERAGE VALUE OF "C" NOMINAL DIAMETER WILLIAMS AND HAZENS FORMULA	PER CENT OF IMPROVEMENT	LIN. FT. OF PIPE LINED	LIN. FT. OF PIPE TESTED	AGE OF PIPE	PIEZOMETRIC FLUID
Durant Avenue, North Charleston:	8″						3450.50	358	27 Yr.	Acetylene Tetrabromide
Before Lining		32.47	350	7.1	53	210		358		" "
After Lining		32.63	351	1.77	112					
Charlotte Street, Washington to Concord:	8″						652.70	257	29 Yr.	Acetylene Tetrabromide
Before Lining		43.26	466	12.9	51	251		257		" "
After Lining		41.28	445	2.125	128					
Prioleau Street, Tradd to S. C. Yacht Club:	6″						688.00	244.7	60 Yr.	Mercury
Before Lining		40.52	436	81.4	39	251		303.0		"
After Lining		36.40	392	12.0	98					
Legare Street, Tradd to South Battery:	6″						1255.00	271.0	60 Yr.	Mercury
Before Lining		34.17	368	37.35	50	234		271.0		Acetylene Tetrabromide
After Lining		35.18	379	8.33	117					
King Street, Broad to South Battery:	6″						2029.10	328.0	60 Yr.	Mercury
Before Lining		34.1	368	46.3	44	257		328.0		Acetylene Tetrabromide
After Lining		36.33	392	9.45	113					
East Bay Street, Broad to Tradd:	8″						600.00	351.0	60 Yr.	Mercury
Before Lining		38.78	417	17.5	40	280		296.0		Carbon Tetrachloride
After Lining		39.45	425	2.51	112					
East Bay Street, Tradd to Water: (No Test)	6″						789.4	—	60 Yr.	—
Before Lining		—	—	—	—	—		206		Acetylene Tetrabromide
After Lining		38.33	418	10.31	113					
East Bay Street, Water to Atlantic: (No Test)	6″						195.6	—	60 Yr.	—
Before Lining		—	—	—	—	—		172		Acetylene Tetrabromide
After Lining		33.6	362	10.85	95					
Hackemann Avenue, Meeting to 4th Street:	4″						1143.54	300	20 Yr.	Mercury
Before Lining		10.77	116	67.9	33	245		300		"
After Lining		26.64	287	69.7	81					
Hampton Avenue, Meeting to 4th Street:	4″						1205.32	286	20 Yr.	Mercury
Before Lining		12.15	130.7	67.3	37	238		286		"
After Lining		29.37	316.4	66.8	88					
Total							12009.16			

Results Obtained

Table 2 gives statistical data as to location, diameter, and condition of the main before cleaning, and after cleaning and lining. Column 6, the reader will note, gives the value of C on nominal diameter of pipe, varying from as low as 33 for 4-inch main before lining, to a maximum of 128 on an 8-inch main after cleaning and lining. The percentage of improvement, as shown by this table, varies from as low as 210 on an 8-inch main to as high as 280 on an 8-inch main. The age of pipe varied from 20 to 60 years. Measurements to determine the value of C were based upon the actual measurement of water through a velocity type of meter, the piezometric head being measured using mercury or acetylene tetrabromide fluid in the U-tube. The length of pipe involved in the tests varied from 172 ft. to a maximum of 358 ft. All of these values are based on the nominal diameter of the pipe as there was no particular interest in the academic value of the actual diameter of the cement-lined main, nor any way of determining the actual diameter of the tuberculated main with which to compare these results. In one case, however, the diameter of the main after lining was measured and the value of C, in this particular case, was 140. This was a 4-inch pipe with actual inside diameter after lining of 3.36-in. the reduction in area being 29.6 per cent. The actual discharge through the 4-inch main before lining was 130,700 gal. per 24 hr. with a loss of head of 67.3 ft. per 1,000 ft. of pipe. After cleaning and lining, with a loss of head of 66.8 ft. per 1,000, the delivery per 24 hr. was 316,400 gal., or an increase in the delivery capacity of 240 per cent.

With the use of hand charging and a high slump (9 to 10 in.), there is considerable semi-laitance material squeezed through the dehydrating orifices so that there is a flat segment left in the bottom of the pipe. This material is of about the same nature as the mortar used in lining the pipe, but nevertheless this segmental area is objectionable to the extent that it does reduce the area of the pipe, and the desirable condition would be to reduce it to the irreducible minimum. To accomplish this, it is desirable to use as low a slump test as possible, and therefore to use a mechanical charging device. Figure 8 shows sections of a 6-inch main before and after cleaning with a section of the same main after lining. Note the segment of semi-laitance in the bottom of the main.

Cost of Project

The cost of the work ran from about $\frac{1}{4}$ to $\frac{1}{2}$ the cost of replacement of main, and of course varied with the kind of paving, number of services, hydrant and street crossing connections. It can readily be seen that the cost of cleaning and lining 8-inch main is relatively much cheaper per foot than the 4- or 6-inch main as the cost of opening up the paving in the street for introducing the cleaning scrapers, cement-mortar and lining machine, service, hydrant, and street crossing connections, is practically uniform without regard to the size of the main lined. The time of cleaning and lining is relatively the same, and therefore the only increased cost in lining 8-inch main over 4- and 6-inch main, is the cost of the material required and the cost of the larger equipment. Actually in the experiments at Charleston, based upon the condition under which the work was done, the cost of the work was as follows:

4-inch main:	
Sand asphalt penetration pavement, with taps averaging one about every 60 ft.	$0.55 per foot
Unpaved street, with few services, little traffic, this cost dropped to	0.45 " "
6-inch main:	
Paved territory, asphalt concrete base, services averaging every 30 ft. apart	0.90 " "
Open territory, unpaved streets	0.46 " "
8-inch main:	
Paved street, asphalt concrete base, with services averaging every 60 ft.	1.55 " "
Unpaved streets, minimum number of services	0.50 " "

These costs included the cost of opening up the street and replacing the paving in every case, and in residential sections where there were numerous services, the installation of a 2-inch galvanized main at the curb, with connections to receive water at each end, and the connection of the service pipes to this temporary main. The cost for the material used in making these temporary connections is not included. The figures given in this tabulation are averaged for this experimental work—the maximum at times probably reached 50 per cent above these averages. This, however, was an exceptional case where the main was 60 years old, was badly deteriorated due to the soil corrosion; and was a very thin cross-section, the walls being less than $\frac{3}{8}$ in. thick in places. On an average the main had a cover of 36 in.

Conclusions

The results as obtained at Charleston have been very satisfactory indeed. There is no doubt that additional lining will be done as soon as financial conditions warrant it. Consumers along the lines which we have lined have been very satisfied in that the volume of the water has been increased and in that they do not suffer from the periodic disturbances and roily water that formerly occurred when there were excessive drafts caused by the flushing of streets and the use of fire hydrants for fire fighting.

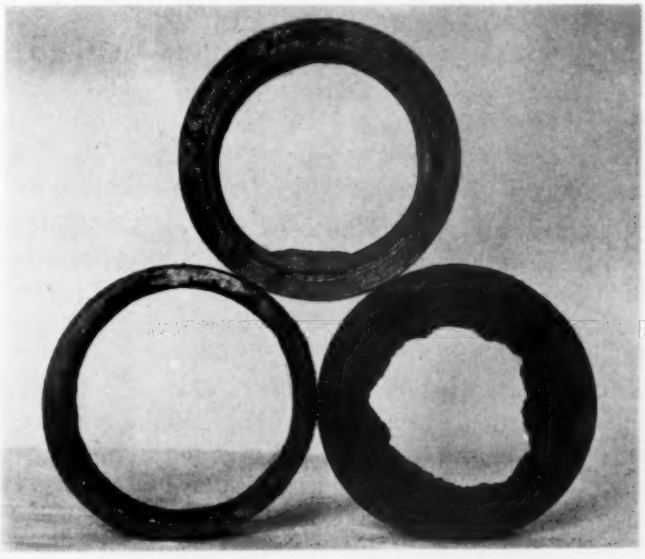

FIG. 8. Sections of 6-inch Main Before and After Cleaning and After Lining

To reduce the segment, or material passing through the dehydrating orifices of the lining machine, it is desirable to maintain as low a slump test as possible and to that extent it is felt that the introduction of some plasticizing material such as Celite, Duracrete, or even hydrated lime would be desirable in that it would give a more plastic and buttery mass than the straight portland cement and sand. The introduction of this inert material would probably slightly reduce the strength of the mortar on a tensile test but it is felt that this is immaterial as long as the mortar has sufficient strength, cohesion, and stability to adhere and maintain the arch in

the main; and experiments show that by the use of such plasticizing material a very much reduced slump test can be had.

Use of Graded or Kiln-Dried Sand

To obtain the best results it is believed advisable to pay considerably more attention to the grading of the sand, even to the extent of having it kiln dried and graded. This was demonstrated conclusively, in the work done by the Tate Pipe Linings, Inc., at Falmouth, Mass., where a kiln dried and graded sand put up by the material contractor in cubic-foot sacks, was used. Analysis of this sand, as furnished by the Tate people is as follows:

Mesh	Percentage Retained	Mesh	Percentage Retained
10	0.5	50	26
20	10	80	5
30	35	100	7
40	16	200	0.5

The advantages of using kiln dried and graded sand are that better control of the cement-water ratio and of the grade of aggregate can be obtained; and as the material is such a small percentage of the total cost, the additional cost of kiln drying and grading is immaterial.

It has been recognized for a number of years that the process of treating and filtering water disturbs the chemical balance of the water, making it more aggressive, and while filtration engineers and chemists have corrected this to a great extent by treatment there is still no definite assurance that in every case tuberculation has ceased. An estimate of the mileage of mains, based upon the 1930 census and the known mileage of mains in some fifty-three cities and urban communities, brought up to July, 1939, indicates that there are in continental United States not less than 224,000 mi. of cast-iron water mains 4 in. and above in size. It is estimated that at least 90 per cent, or 200,000 mi., are from 4 to 12 in. in diameter, so it can readily be seen that with the known active or aggressive waters throughout the country there is a large field and necessity for the restoration of carrying capacity of the mains now laid and in service. The reconditioning of these water mains in position means that their functional life will be increased from a few years to probably as many as fifty, depending upon their present age, that a great economic saving to the water companies and municipalities throughout the United States will be effected.

Permanency of Lining

There has been considerable discussion among engineers and chemists as to the permanency of cement-mortar lining, particularly due to the leaching or dissolving out of the calcium content of the cement, destroying the stability of the mortar. Time, which is the test of all material things, indicates that this deterioration is, much like Mark Twain's death, "very greatly exaggerated" for we have records of cement-mortar lined water mains and service pipe laid in New England, 60 years of age or older, that are still in service and going strong.

The Tate Company reports that it has lined, experimentally, mains as large as 15 in. in diameter. There is no reason why lining in position cannot be carried out successfully on pipe up to 12 in. in diameter. The difficulty of lining larger mains by this process is in maintaining a satisfactory plug or volume of cement mortar to fill the cross-section of the main in advance of the lining machine. There is no doubt, however, that inventors will devise some ingenious method of maintaining some form of piston in front of the mortar to maintain a full pipe of mortar and prevent the escape of the mortar in advance of the machine except as required.

There are, as far as we are aware, only two companies attempting to recondition or line mains in position: the Tate Pipe Linings, Inc., of Andover, Mass., for mains from 6 to 12 in. in diameter, and the Centriline Corporation of New York, a new company that has recently been organized by the Gillespie Company, the Lock Joint Pipe Company, and the Raymond Concrete Pile Company, who have taken over the patents of the Perkins Process for lining steel and cast-iron mains of large diameter with cement mortar. This work has formerly been done by the T. A. Gillespie Company.

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Testing Motor-Driven Pumps

By R. W. Angus

IN MUNICIPAL pumping stations, centrifugal and turbine pumps have almost entirely replaced the earlier reciprocating type. As a rule, the reciprocating or plunger pumps were steam driven, and guarantees for efficiency were usually expressed in millions of foot pounds of work done on the water per 1,000 lb. of steam or per 100 lb. of coal used. Though extremely accurate tests required very great care in observations, approximate results could readily be obtained because the steam could be condensed or the coal weighed, and the work was always based on the plunger displacement which could be calculated from the speed and the plunger diameter and stroke.

Centrifugal or turbine pumps are invariably directly connected to electric motors, without speed reduction; and guarantees on them are usually given on the basis of the discharge at a given delivery pressure. Sometimes the pump must give stated approximate deliveries at pressures above and below the guaranteed best efficiency condition. The suction conditions are sometimes described in detail, but most often they are not.

In addition to the water pumped, efficiency must also be guaranteed, and this is sometimes stated as the overall wire-to-water efficiency, which is the product of the motor and pump efficiencies. At other times the guarantee is required on the motor and the pump separately. The motors are usually of the induction type, with speed varying slightly with the output, but synchronous motors of fixed speed are also used.

It is therefore evident that the testing of a centrifugal pump requires measurements of the volume of water discharged, the suction

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and discharge pressures, the speed of the unit, the electrical input, and possibly the electrical losses; but unless the specifications are carefully drawn, there is considerable uncertainty in the determination of the various quantities, and corresponding uncertainty with regard to the fulfillment of the contractor's obligations. Starting first with the pressures on the pump, suction and discharge pressures are usually measured by attaching gages to the corresponding flanges of the pump and allowing for difference of elevation between the gages or points of attachment. In addition, allowance should be made for difference in velocity heads in the suction and discharge pipes. Some engineers neglect to mention this item in drawing the specifications, but, then, the intent becomes uncertain.

Errors from Gage Connections

In a previous paper read before the Association and published in the JOURNAL for April, 1937, the writer called attention to the errors that may arise from wrong gage connections, and showed that if four accurate gages are attached 90 degrees apart on the same flange, the pressures indicated by them may differ as much as 3 or 4 ft., depending on the conditions inside the pipe in proximity to the measuring opening. The pipes must be smooth, clean and free from projections or disturbing lumps of metal on the inside, and the gage connections should be made at four points 90 degrees apart, the four connections being attached to a fairly large pipe around the flange. This surrounding pipe should have a connection for the gage. This precaution is rarely taken in any but the largest pumps, but if only one pressure hole is to be drilled through the pipe, it should be 3 in. or 4 in. from the face of the flange.

Suction lifts or pressures may usually be best taken by mercury column, because the column need rarely exceed 24 in. in length, whereas a reliable spring gage usually is most convenient for discharge pressures. These gages must be very carefully calibrated by a dead-weight gage tester before and after the test, and should previously have been in service long enough for the spring to have established a stable condition. It is sometimes possible that both the suction and delivery pressure measurements may be made by mercury columns. This is by far the most accurate method.

In the recent tests of the pumps at the Victoria Park Toronto Water Works, mercury columns were used for all pressure measurements, even for heads of 270 ft., in which case the mercury column

was nearly 25 ft. high. In this case the accuracy was very much improved in some of the pumps by connecting the suction and discharge pipes to the mercury column in such a way that the latter read the net pressure rise produced by the pump and automatically allowed for difference of elevation of the connections. The results so obtained represent a high degree of precision.

In most pumps the rate of discharge must be determined by a Venturi meter. In this case one should not rely upon the dial readings if great accuracy is required, because the pointer frequently has a relatively long period of swing and, further, because there may be some errors in the mechanism connecting the floats and indicator. It is always much more reliable to measure the differential across the tube directly, either by using a mercury differential gage or, preferably, by reading directly the differential in feet of water on a suitable gage. A mercury differential gage may be connected across the meter close to the recorder as there is a $\frac{3}{8}$-inch tapped and plugged hole at the back of the recorder on both connections. This may be used for calibrating the recorder, but it is also convenient for connecting the differential gage.

The differential gages may be purchased from meter builders, or they can be made up in any machine shop at no great expense by arranging two headers with stuffing boxes so that two parallel glass tubes of suitable length may be fastened between them. The bore of the tubes should be about $\frac{3}{8}$ in., and they need not be of heavy glass, as very thin walls may be used on high pressures for cold water. The most serious danger is bending or twisting the tubes, a trouble to be avoided only by fastening the two headers rigidly to a steel bar or wooden board. All parts must be made of steel or iron, as mercury attacks all yellow metals.

The flow of water is calculated from this differential reading by the formula (given below) which is based on the assumption that the connecting piping has been well cleared of air and that all of the tubing between the mercury and the meter is full of water. If M is the difference in inches between the two columns, and G is the corresponding number of Imperial gallons per minute, then

$$G = 3080 \frac{A_1 A_2 c \sqrt{M}}{\sqrt{A_1^2 - A_2^2}}$$

where A_1 and A_2 respectively are the areas of the upstream end and the throat of the meter in square feet.

A similar formula may be used for checking up the indicator on the recorder, only in that case the connections to the tube may be closed off and an artificial differential produced across the two sides of the recorder by glass tubes attached, as in the above case, and filled with water to different levels. If the differential H is measured in feet of water, then

$$G = 3000 \frac{A_1 A_2 c \sqrt{H}}{\sqrt{A_1^2 - A_2^2}}$$

The coefficient c (which must be selected to suit the meter) has a value of 0.983 for a 14x9.5-inch meter, going up to 0.988 for a 42x27-inch meter. Exact values of the constant have been adopted by agreement and can be had from the meter builder.

The electrical measurements are sometimes taken from switchboard instruments, but these are not satisfactory if a high degree of accuracy is desired. A set of portable instruments, including voltage and current transformers, ammeters, voltmeter and a polyphase wattmeter are necessary. The transformers should be of such ratio that the wattmeter and ammeters have as nearly full scale reading as possible, although at times one must work at half scale or less. For safety, the secondaries of all transformers should be grounded.

If the efficiency of the pump is to be measured separately, it is necessary to determine the motor losses, a rather difficult thing to do at the pumping station. Some of the measurements, such as light load power, may be made by disconnecting the motor, and if direct current is available, the resistances may be measured, but one may have to rely on the motor builder for such information.

In electrical systems, such as the Niagara one, where the frequency is maintained constant within very narrow limits, the speed of the machine is best measured by subtracting the slip from the synchronous speed. The slip is easily measured by a neon lamp and a cardboard attached to the coupling, this cardboard having one or two radial slots in it. By standing some distance from the cardboard in such a position that the part with the slots in it is between the observer and the neon light, it is quite easy to count the slip by the aid of a watch, as it is measured by the number of times the light is seen per minute.

Tests should always be made under conditions given in the specifications, particularly with regard to the suction side, as the perform-

ance of pumps may vary a great deal with slight suction changes. The suction lift should not exceed that specified. It is also advisable to work to a discharge above that specified and to go down to about half the specified delivery, also to take a reading with the delivery valve closed. The latter should be done as quickly as possible, as the condition is not a good one for the pump.

Tests Made at Toronto

During the past summer the writer had the pleasure of conducting the acceptance tests on the pumps at the new Victoria Park Pumping Station in Toronto. The tests were done on instructions from R. C. Harris, Commissioner of Works, and with the kindly co-operation of A. U. Sanderson and L. F. Allen of the Works Department. There are eleven pumps in this station, three for supplying wash water for the filters, four for pumping raw water from the intake well to the filter beds, and four for delivering filtered and purified water to the distribution system. Each pump is coupled to its own motor supplied by 25-cycle, 3-phase, 2,200-volt current.

The three wash water pumps have a net head of 54 ft. and a capacity of 5, 7½ and 10 m.g.d. (Imp.), being driven respectively by 75, 100 and 140 h.p. induction motors at 730 r.p.m.

The four raw water pumps work at a head of 82 ft. and are for 20, 25, 40 and 50 m.g.d. (Imp.), respectively. They are driven by synchronous motors of 425, 525, 825 and 1,025 h.p. There are two 5 mil. gal. filtered water pumps for a 196-foot head, driven by 225 h.p. synchronous motors, and two 25 mil. gal. filtered water pumps for a 270-foot head driven by 1,700 h.p. synchronous motors.

The aggregate pumping capacity of the station is 217.5 m.g.d. (Imp.), and the total power of the driving motors is nearly 7,000 h.p.

While the station deserves some study on account of its large capacity, the pumps are of unusual interest because of the very high efficiencies guaranteed on them, these efficiencies covering the overall wire-to-water results. The guarantees for the wash water pumps ranged from 77.9 to 80.5 per cent overall efficiency, those for the raw water going from 84.5 per cent for the smallest one to 86.3 per cent for the largest one, while the filtered water units were guaranteed at 81.4 per cent for the 5 mil. gal. unit and 86.0 per cent for the 25 mil. gal. unit. When the motor efficiency is taken into account, this means that the latter pump exceeds 90 per cent efficiency.

In order to get accurate results, the contractors, Dominion Engi-

neering Works, Montreal, made model pumps to represent each unit. These models were carefully studied in the testing laboratory and then used as the basis for the full sized pump, a method which proved very satisfactory. In making the acceptance tests on the units in

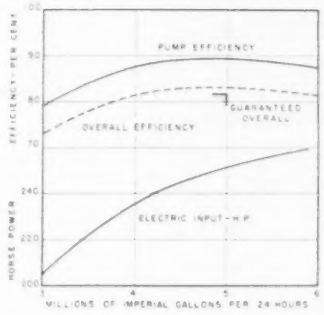

FIG. 1

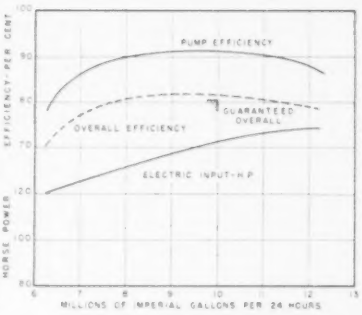

FIG. 2

FIG. 1. Pump for 5 mil. Imp. gal. per day at 196-foot head and 750 r.p.m., Toronto Water Works

FIG. 2. Pump for 10 mil. Imp. gal. per day at 54-foot head and 730 r.p.m., Toronto Water Works

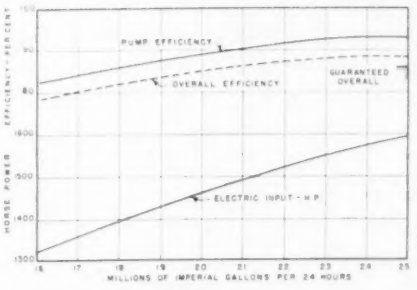

FIG. 3

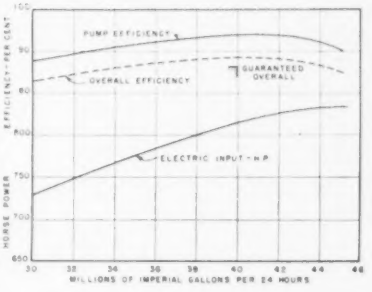

FIG. 4

FIG. 3. Pump for 25 mil. Imp. gal. per day at 270-foot head and 750 r.p.m., Toronto Water Works

FIG. 4. Pump for 40 mil. Imp. gal. per day at 82-foot head and 500 r.p.m., Toronto Water Works

the station, it was necessary, of course, to take extreme care to get reliable results so pressures were balanced by mercury columns. The differential on the Venturi meter was read directly in feet of water and the electrical readings were from instruments carefully calibrated against standards. To get rid of any personal error in

readings, all instruments were photographed simultaneously and the photographs read later under a microscope. In this way a clear and definite reading was obtained in each case.

The performance of the pumps at the tests reflected marked credit on the engineers of the Dominion Engineering Works. That the very high efficiency guarantees were exceeded is shown by typical curves on four of the pumps (Figs. 1, 2, 3 and 4), the results on other pumps being omitted to save space.

Since water works engineers are particularly interested in the pump efficiencies, the two curves of overall efficiency and pump efficiency have been plotted in each case, as well as the curve of horse power input to the motor. To avoid complicating the diagrams, the head curves have been omitted. All of the pumps showed remarkably high efficiencies and maintained these efficiencies over a fairly wide range of discharge.

The writer is indebted to R. C. Harris, Commissioner of Works, Toronto, for permission to give data on the Victoria Park tests; also to A. U. Sanderson and L. F. Allen of the Works Department, who came more directly in contact with the details. The writer is also grateful to H. S. Van Patter, Hydraulic Engineer, and particularly to H. Ulmann, Pump Designer, both of the Dominion Engineering Works, for their co-operation and courtesy during all of the work.

Methods of Testing Pumping Equipment

By C. J. Des Baillets

THIS paper contains a simple explanation of pump testing and describes in detail three pump tests representing the standard testing practice of the Montreal Water Board.

The fundamental formula for pump efficiency is:

$$\text{Efficiency} = \frac{\text{Hydraulic Power Developed by Pump}}{\text{Power Supplied to the Pump}}$$

The power supplied to the pump may be measured in brake horsepower (b.h.p.) or in kilowatts (kw.).

The hydraulic power developed by the pump is the product of the pump discharge, Q, by the head, H.

In the efficiency formula, the product of Q, H, and b.h.p., or kw., must be expressed in the same units, generally in ft.-lb. per sec. This necessitates the introduction of a coefficient in the formula, the coefficient depending on the units used in the measuring of H, Q, b.h.p. or kw.

Table 1 gives the efficiency formulas with the different coefficients depending on the unit used for Q, when H is in feet.

The problem is to measure: (1) the quantity of water pumped (Q), (2) the head developed by the pump (H), and (3) the power supplied to the pump (b.h.p. or kw.). The pumps are rated for a specified number of revolutions per minute. If the test is run at a different speed, the measured values of Q, H and b.h.p. or kw. must be corrected.

Three methods were used in tests by the Montreal Water Board: (1) by Venturi meter, (2) by volumetric measurement, and (3) by a weir. These three methods are described in detail in the description of the tests.

A paper presented on March 27, 1940, at the Canadian Section Meeting at London, Ont., by C. J. Des Baillets, Chief Engineer, Water Board, Montreal.

The total head in a pump is measured in feet of water by the difference of elevation of the tops of water columns in tubes open to the atmosphere, and so installed as to record the pressure of each of two sections AA and BB of the piping of equal diameter, the one on the suction side and the other on the discharge side of the pump. It is shown by the symbol H in Fig. 1 and is equal to $H_1 + H_2$ (Case a).

TABLE 1

Pump Efficiencies

Q MEASURED IN	POWER SUPPLIED TO THE PUMP IN	
	b.h.p.	kw.
Gallons per minute (Imperial)	$\frac{Q \times H}{3300 \times \text{b.h.p.}}$	$\frac{Q \times H}{4424 \times \text{kw.}}$
Cubic feet per second	$\frac{Q \times H}{8.828 \times \text{b.h.p.}}$	$\frac{Q \times H}{11.83 \times \text{kw.}}$
Million gallons daily (Imperial)	$\frac{Q \times H}{4.752 \times \text{b.h.p.}}$	$\frac{Q \times H}{6.370 \times \text{kw.}}$

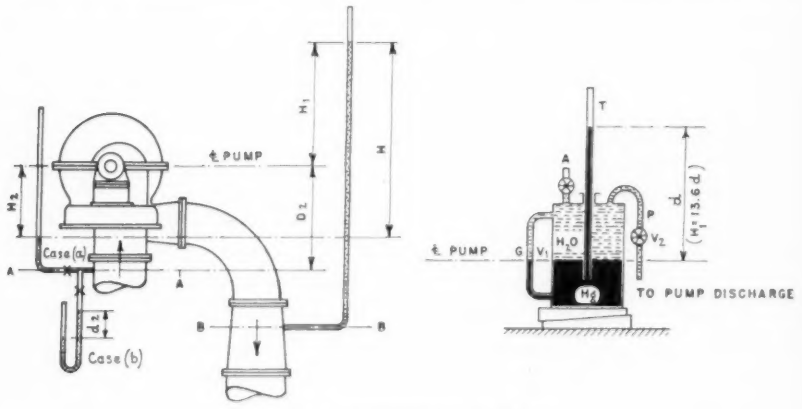

FIG. 1. Measurement of Head

On the suction side, the pressure may be below atmospheric. In this case, $H_2 = D_2 + d_2$ (Case b).

Should it be impracticable to have the two sections AA and BB of equal diameter, there must be added to the head measured as above, the velocity head corresponding to the difference of velocities at sections AA and BB.

When the head on the discharge side cannot be measured conveniently by a water column, a mercury gage or a calibrated pressure gage must be used.

At sections where the pressure is measured, four piezometer openings should be made at opposite ends of two diameters at right angles, and joined by a ring to which the gage is connected. Provision should be made to prevent air from accumulating at any point of the connecting pipes. The effect of the temperature on the gages cannot always be discounted.

As most of the motors operating pumping units in this country are alternating current 3-phase motors, the method of measuring the electrical energy absorbed by the motor will be called the "polyphase wattmeter method" when a polyphase wattmeter is available, or the "two single-phase wattmeter method" when two of such meters are available.

For voltage up to 550 volts, current not exceeding 10 amperes, direct reading wattmeters may be used; but for voltage and current exceeding these values, instrument transformers will be required for both potential and current.

The accuracy of the test depending on the accuracy of the meters, it is desirable that all instruments and instrument transformers be calibrated with standard meters in the laboratories of Canadian universities both before and after the test. A correction curve being given for each instrument, the proper corrections will be made to each reading before recording them in the tables of tests.

In the two single-phase wattmeter method, (see electrical measurement of Test 1) the power used by the meter will be determined by the arithmetical sum of the readings: $W = W_1 + W_2$, and the power factor will be determined by the formula:

$$\text{P.F.} = \frac{\sqrt{3}\,(W_1 - W_2)}{W_1 - W_2}$$

In the polyphase wattmeter method the power is given directly by the instrument reading W and the power factor will be obtained for a balanced circuit by the formula:

$$\frac{\text{Wattmeter reading}}{\sqrt{3} \times \text{volts} \times \text{ampere of two balanced phases}}$$

If the phases are unbalanced the power factor will have to be determined by a power factor meter (see Fig. 2).

The total kw. input, as determined by the wattmeter methods, multiplied by the motor efficiency will give the energy applied to the pump shaft. In order to express this energy in horse power, the kw. will be divided by a constant: $W \div 0.746 = \text{h.p.}$

In tests where the flow is measured by any kind of a meter (Venturi, weir, orifice, etc.), the procedure should be as follows:

After obtaining a steady discharge of any desired amount, readings are made by a group of observers of the various quantities necessary to determine the efficiency. For each steady discharge, ten observations, preferably, should be made at regular intervals of each of the

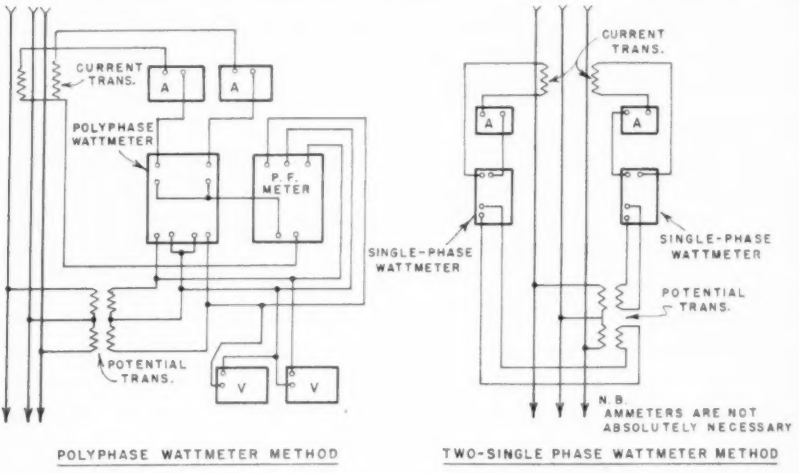

FIG. 2. Electrical Measurement

quantities involved. All the readings must be as simultaneous as possible.

Each group of observations is called a "run" and the average value of the group is used in determining the efficiency at the corresponding discharge.

The condition of the steady flow cannot always be obtained when the discharge is measured by a volumetric method.

Let us now consider three tests made at the plants of the Montreal Water Works, under the supervision of the Montreal Water Board.

The tests differ principally in the method used to measure the quantity of water pumped. In Test 1, pump discharge was measured by Venturi meter; in Test 2 by volumetric measurement; and in Test 3 by the weir method.

In Test 1 (Fig. 3) the pumps to be tested were rated 30 m.g.d (Imp.)* each, at 240 ft. head. For station records, a 36 x 24-inch Venturi meter was installed in each discharge line and used for the tests, the flow being read on a differential manometer graduated in m.g.d.

At the rated capacity of the pumps, the differential head on the manometer was only 3.5 in. of mercury, and a difference of 1 m.g.d. was represented by $\frac{1}{4}$ in. of the scale. The flow could not be estimated closer than 0.10 in 30, or one-third of 1 per cent.

If the throat of the Venturi meter had been smaller, the difference

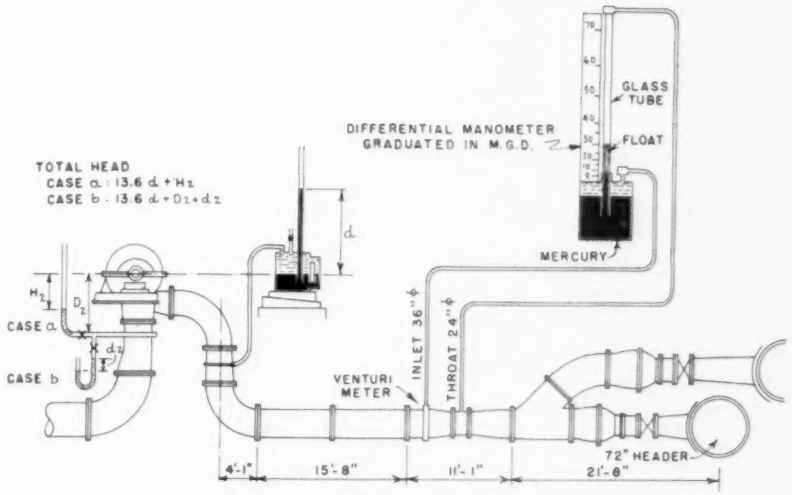

FIG. 3. Test 1—Venturi Method

of pressure would have been greater and the scales could have been read with greater accuracy. Such a reduction in the throat diameter, however, would have meant a higher loss of head in the meter and consequently a continuous waste of power in operation.

The Venturi meter is based on the principle of the conservation of energy. If

Q = flow through the meter,
A = sectional area of main at inlet,
V = mean velocity in main,
a = sectional area of throat, and
v = mean velocity at throat, then
$Q = AV = av$.

*Note that calculations are on basis of Imperial gallons.

The difference of pressure between main and throat,

$$\frac{v^2}{2g} - \frac{V^2}{2g} = H_v$$

is the Venturi head:

$$Q = A \sqrt{\frac{2g \times H_v}{\frac{A^2}{a^2} - 1}}$$

The value H_v is measured by means of a differential manometer. This measured value of H_v is higher than the theoretical differential head $\frac{v^2}{2g} - \frac{V^2}{2g}$. This is due to the friction losses between the inlet and the throat and to departures from the theory based on mean velocities.

Therefore we have:

$$Q = kA \sqrt{\frac{2g \times H_v}{\frac{A^2}{a^2} - 1}}$$

The value of the coefficient k is often given by the manufacturers of Venturi meters as 0.99. This is considered high; $k = 98$ is probably nearer the true value, and $k = 97$, sometimes used, would be conservatively on the low side. Any change in the coefficient involves a corresponding change in the efficiency.

In view of all the conditions, it is questionable whether the discharge of the pumps is measured to within 1 per cent.

The standard method shown in Fig. 1 was used in testing the head developed by the pump. The discharge head was measured by a mercury column in the form of a pot gage. Wedges were used to bring the level of the mercury in the pot to the center line of the pump (see Fig. 3).

Water columns in glass tubes were used to measure the suction lift which, however, is a small quantity relative to the discharge head. As discharge pressure and suction lift connections were made at sections of the piping of equal diameters, there was no correction for velocity head.

In the electrical measurement tests both polyphase and two single-phase wattmeter methods were used simultaneously and the average power obtained was used in the efficiency formula (see Fig. 4). Calibration curves for instrument transformers are shown in Figs. 5 and 6.

As this pump was driven by synchronous motor, there was no correction for a pump speed different from the rated r.p.m. The speed was checked by a direct reading tachometer.

In this case the overall efficiency of the pumping unit had been specified and guaranteed by the pump manufacturer, and the unit was tested for combined efficiency (see Table 2 and Fig. 7).

The test was carried out, under the supervision of the Montreal Water Board, by Dean E. Brown, Professor of Applied Mechanics and Hydraulics, McGill University, and Theo. J. Lafrenière, Professor of Sanitary Science, in charge of Hydraulics, École Polytechnique.

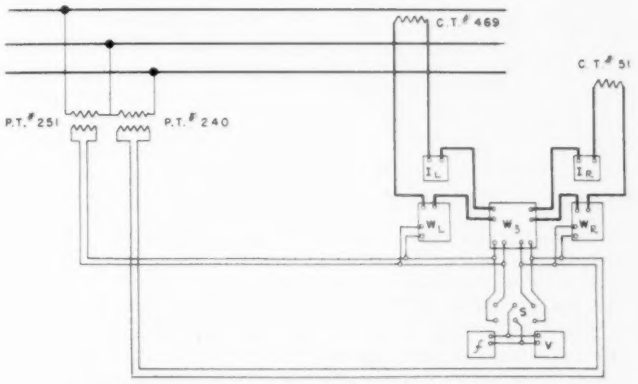

FIG. 4. Diagram of Electrical Connections

W_L and W_R are single-phase wattmeters. W_3 is poly-phase wattmeter. I_L and I_R are ammeters. V is volt-meter. f is frequency meter. S is double-pole, double-throw switch.

Test 2 is an example of the measurement of the output of the pump by the volumetric method (see Fig. 8).

The washwater pumps to be tested were rated 4,000 g.p.m. against a head of 65 ft. and discharged in a tank 42 ft. in diameter and 12 ft. deep. For the purpose of the test, the discharge pipe was extended up to the high water elevation in the tank. Thus the head could be kept constant during each test run. This extension was removed after the pumps had been tested.

It was not convenient to empty the tank during the test. By pumping about 15,000 gal. per run, only 5 points for the efficiency curve could be obtained. The different discharges at which the runs were to be made were chosen beforehand. Preliminary tests gave

the exact amount of throttling of the discharge valve necessary to get the required discharge.

The duration of each run was noted by use of a stop watch. The time used in the computations was one second less than the recorded time to allow for time taken by motor to reach maximum speed.

The output was further corrected by the addition of 50 gal. per min. to account for the losses at the valves in the filtration plant. These are the measured losses.

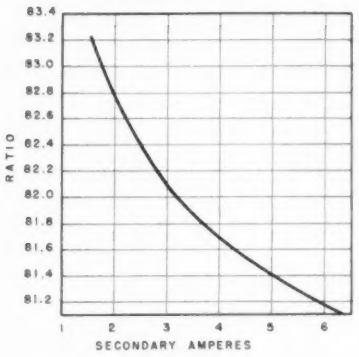

FIG. 5

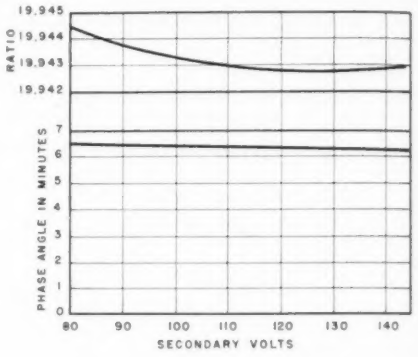

FIG. 6

FIG. 5. Calibration Curve for Series Transformer (Standardizing Laboratory, Electrical Eng. Dept., McGill University)

Series transformer, 400 to 5 amperes through type. Secondary burden—two wattmeters and one ammeter, all 5-amp. range, plus 60 ft. of No. 14 B & S copper.

FIG. 6. Calibration Curve for Potential Transformer (Standardizing Laboratory, Electrical Eng. Dept., McGill University)

2200/1100 to 110 volts, 15 volt-amps. Secondary burden—two wattmeters, equal to 3539 ohms.

The head on the discharge side of the pump was measured by a mercury pot gage as in Test 1. The suction lift was measured by a mercury gage.

In the electrical measurements, the polyphase wattmeter method was used.

As the pump speed varied with the load, it was checked with a direct reading tachometer. All readings had then to be corrected to the rated r.p.m. Corrections were made on the following basis:

The discharges are directly proportional to the speeds. The heads

are proportional to the square of the speeds. The power input is proportional to the cube of the speeds.

The efficiency of the motor was taken as 90.5 per cent as given by the motor manufacturer.

TABLE 2

Centrifugal Pump Test

Test No.: *1* Date: *21 Sept. 1933*
Pump Make: *Worthington* Serial No.: Size: *32″ x 21⅜″*
Capacity: *30 m.g.d.* Head: *240 Ft.* R.p.m.: *600*
Capacity Measurement: *Venturi Meter* Meter Coefficient: *.99*
Discharge Head: *Mercury Column* Suction Lift: *Water Column*

READING NO.	HYDRAULIC MEASUREMENTS					ELECTRICAL MEASUREMENTS			COMPUTED OVER-ALL EFFICIENCY OF PUMPING UNIT
		Head				Kilowatts Used			
	Q Capacity, m.g.d.	Discharge		Suction Ft. Water	H, Total Head, Ft.	Polyphase Wattmeter, kw.1	2 single-phase Wattmeters, kw.2	Kw. Used for Efficiency. Mean of kw.1 and kw.2	
		Ft. Hg	Ft. Water						
1	19.00	19′—10.125″	269.75	3.87	273.62	1038.7	1034.6	1036.6	78.7
2	20.98	19′— 5.373″	264.42	4.90	269.32	1100.0	1096.1	1098.1	80.8
3	25.19	18′— 9.400″	255.41	4.77	260.18	1216.4	1217.7	1217.0	84.5
4	27.88	18′— 2.250″	247.31	5.12	252.43	1281.8	1281.4	1281.6	86.2
5	30.13	17′— 6.750″	238.81	5.51	244.32	1323.6	1322.4	1323.0	87.3
6	31.25	17′— 2.540″	234.51	5.71	240.22	1351.5	1347.7	1349.6	87.3
7	31.87	17′— 0.662″	231.81	5.83	237.64	1354.3	1353.0	1353.7	87.8
8	32.96	16′— 8.155″	226.80	6.02	232.82	1375.8	1375.4	1375.6	87.6
9	33.97	16′— 3.693″	221.80	6.25	228.05	1393.4	1388.4	1390.9	87.4
10	34.88	15′—11.829″	217.31	6.42	223.73	1406.1	1403.2	1404.7	87.2
11	35.40	15′— 9.050″	214.10	6.53	220.63	1409.8	1406.5	1408.2	87.1

$$\text{Pumping Unit Efficiency} = \frac{Q \times H}{6.370 \times \text{kw.}} \times 100$$

Motor Make: *Westinghouse* Type: *Synchronous*

Table 3 and Fig. 9 show test results of the pump efficiency. This test was carried out by E. Zbinden, Mechanical Engineer at the Montreal Water Board.

In testing three 50 m.g.d. low-lift pumps for the filtration plant, the method of measuring the flow over a weir was found the most suitable. The general arrangement for this test, which is Test 3, is shown in Fig. 10.

At one end of the discharge chamber an overflow was provided in

TABLE 3

Centrifugal Pump Test

Test No.: *2*
Pump Make: *Can. Allis Chalmers*
Capacity: *4000 g.p.m.*
Capacity Measurement: *Calibrated Tank and Stop Watch*
Discharge Head: *Mercury Column*

Serial No.: *A-1479*
Head: *65 Ft.*

Date: *30 Oct. 1936*
Size: *14" x 14"*
R.p.m.: *860*

Suction Lift: *Mercury Gage*

READING NO.	HYDRAULIC MEASUREMENTS										ELECTRICAL MEASUREMENTS		EFF. %	R.P.M.	CORRECTION FOR R.P.M. 860		
	Head						Capacity										
	Discharge		Suction			H Total Head, Ft. Water	Height in Tank, Ft.	Total Pumpage, Gallons	Time (Corrected) Min.	Q g.p.m. Corrected	Motor Input, kw.	Power Supplied to Pumps, b.h.p.			Q, g.p.m.	H, Ft.	b.h.p.
	Ft. Hg	Ft. Water	h_1 Ft. Hg	h_2 Ft. Hg	Ft. Water												
1	4.11	55.9	1.198	.676	5.9	61.8	1.75	15350	2.98	5210	91.5	110.7	87.5	865	5190	61.1	108.2
2	4.33	59.0	1.170	.694	5.3	64.3	1.89	16550	3.48	4800	88.6	107.2	87.2	865	4780	63.6	105.0
3	4.77	65.0	1.109	.756	3.7	68.7	1.79	15700	3.99	3980	81.4	98.5	84.2	870	3940	67.4	95.5
4	5.19	70.6	1.053	.809	2.3	72.9	1.73	15200	5.00	3090	74.0	89.5	76.1	870	3060	71.4	86.6
5	5.59	76.0	1.011	.853	1.1	77.1	1.94	16780	8.00	2150	68.0	81.3	61.8	875	2110	74.8	77.7
6	6.09	82.8	.946	.917	−.5	82.3	0	0			50.9	61.6	0	875	0	79.8	59.0

Motor Efficiency Taken as 90.5%

Pump Efficiency $\dfrac{Q \times H}{3300 \times \text{b.h.p.}} \times 100$

Motor Make: *General Electric*

Type: *Squirrel Cage*

the shape of a trough 9 ft. wide, 7 ft. high and 25 ft. long. By leaving an 8-foot panel in each side wall unconcreted, water could be admitted to the trough at the end at which the weir was erected. The faces of the trough and the upstream face of the weir were cement plastered smooth and true, as perfectly parallel as possible, and at 90 degrees to the upstream face of the weir. Particular care was given to the setting and leveling of the weir edge.

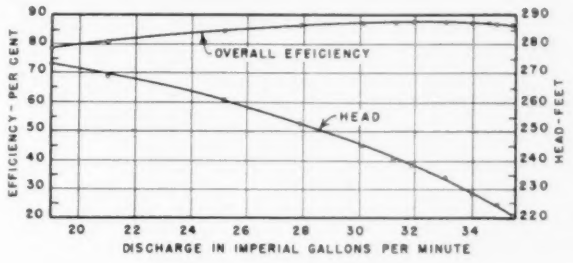

FIG. 7. Results of Test 1

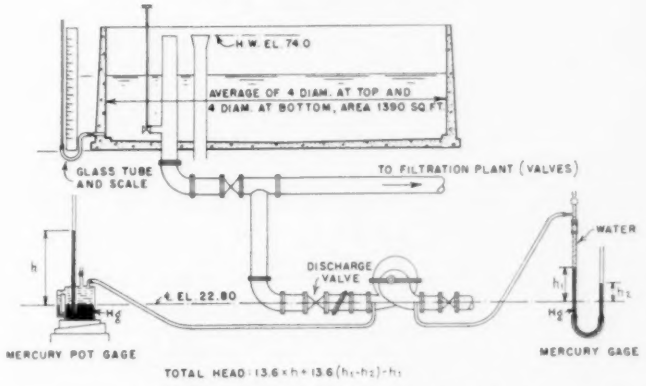

FIG. 8. Test 2—Volumetric Method

Upstream from the weir, the water was stilled by wooden baffles placed vertically in rows across the weir channel. The head on the weir was read on two glass gages placed on the front of a scale in the pump room. The two gages were connected to each side of the trough by rubber hose and 2-inch standard pipes.

The aeration of the underside of the sheet of water falling over the weir was effected by means of a 9-inch diameter opening, the four 3-inch pipes, which had first been provided, having proved insufficient.

A head discharge curve was prepared for the weir. It was plotted from the mean values of the discharges computed by five of the best known formulas (Fig. 11). Comparing the curve adopted and the values obtained from the formulas, the greatest difference is about $1\frac{1}{2}$ per cent above and below the mean value. It should be borne in mind that, at the rated capacity of the pump, an error of .01 ft. in

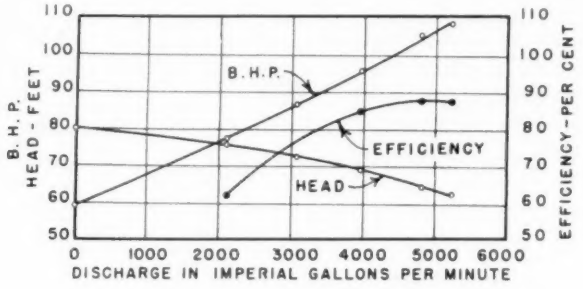

FIG. 9. Results of Test 2

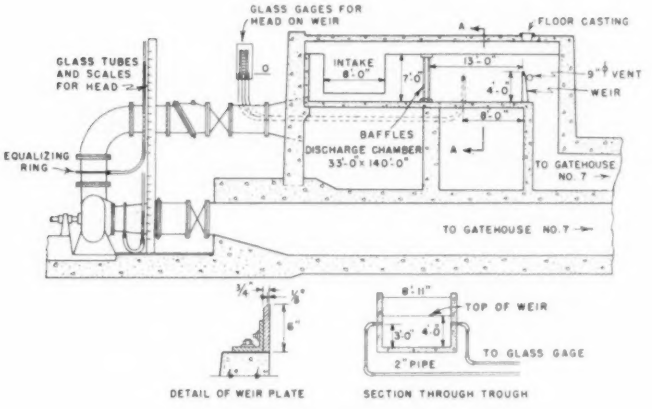

FIG. 10. Test 3—Weir Method

the reading of the weir head means a difference of only 0.6 cu.ft. per sec. or an error in the efficiency of about 0.7 per cent.

Both suction and discharge heads were read on water columns in glass tubes placed in front of a graduated scale.

The power input of the motor as recorded in Table 4 is the average of power obtained by the two methods as in Test 1. As in Test 1, this pump is driven by a synchronous motor. The normal frequency of the power system is 60 cycles per second. The frequencies at the

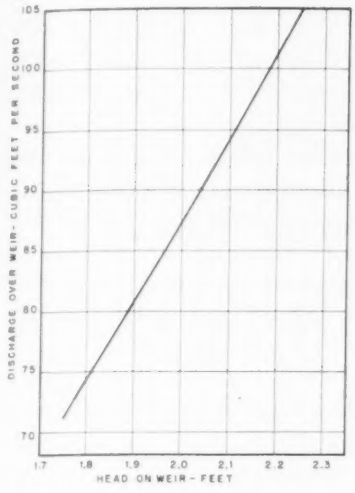

FIG. 11

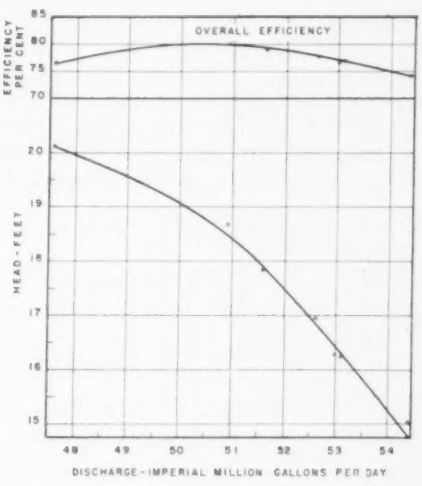

FIG. 12

FIG. 11. Head-Discharge Curve for Weir Tests
FIG. 12. Results of Test 3

TABLE 4

Centrifugal Pump Test

Test No.: *3* Date: *16 Nov. 1936*
Pump Make: *Dominion Engineering* Serial No.: Size: *40" x 42"*
Capacity: *50 m.g.d.* Head: *18.5 ft.* R.p.m.: *400*
Capacity Measurement: *Weir* Discharge Head: *Water Column* Suction: *Water Column*

READING NO.	HYDRAULIC MEASUREMENTS							ELECTRICAL MEASUREMENTS	OVERALL EFFICIENCY OF PUMPING UNIT
	Head			Capacity					
	Discharge El.	Suction El.	H Head, Ft.	Left Gage Glass, Ft.	Right Gage Glass, Ft.	Average Weir Head, Ft.	Q Discharge, m.g.d.	MOTOR INPUT, KW.	
1	46.758	31.760	15.00	2.192	2.196	2.194	54.4	171.3	74.6
2	46.750	30.500	16.25	2.162	2.164	2.163	53.1	176.0	76.8
3	46.779	30.509	16.27	2.161	2.161	2.161	53.0	175.6	76.9
4	46.675	29.734	16.94	2.147	2.150	2.149	52.6	178.8	78.0
5	46.633	28.793	17.84	2.117	2.124	2.121	51.6	182.5	78.9
6	46.540	27.859	18.68	2.101	2.108	2.105	50.9	186.3	80.0
7	46.724	26.508	20.12	2.011	2.015	2.013	47.6	196.0	76.5

Pumping Unit Efficiency $\dfrac{Q \times H}{6.370 \times \text{kw.}}$

Motor Make: *Westinghouse* Type: *Synchronous*

times of each set of readings were checked from the graphic records of the Montreal Light, Heat & Power Cons. system, which read to one-tenth of a cycle. No variation greater than 0.15 of a cycle from the standard of 60 cycles was recorded.

The unit was tested for overall efficiency for, although the pump efficiency and the motor efficiency had been specified separately, the pump manufacturer guaranteed the combined efficiency (see Table 4 and Fig. 12).

This test was carried out under the supervision of the Montreal Water Board, by Dean E. Brown and Theo. J. Lafrenière.

Conclusions

In any pump test, the choice of the gages and the methods of measuring the flow are principally a matter of convenience and opportunity. The key to a successful pump test lies in the care taken to eliminate all possible source of error. It is impossible to list all the factors that may affect the accuracy of the test.

It is to be noted that the efficiency is directly proportional to the coefficients used in the formulas for the measure of the flow, and the accuracy of the test is no higher than the accuracy of these coefficients.

It must also be remembered that the problem of testing a pump starts with the design and the construction of the plant; and making proper provision in the design of the piping, both on suction and discharge side, as well as incorporating measuring devices such as tanks, meters, weirs, etc., if possible, will greatly facilitate the test, add to its accuracy and make a success of it.

The co-operation of the pump manufacturer with the purchaser will, however, be instrumental in achieving these ideal conditions.

Photo-Cell Control of Water Chlorination

By J. H. Harrington

OVER fifteen years ago the writer's attention was directed toward a search for some dependable means, applicable to the process of sterilization of water by chlorine, for closer regulation of the chlorine dosage to a greatly variable biochemical demand than was possible by manual control alone.

Montreal's water supply has always consisted of variable mixtures of the waters of the St. Lawrence and Ottawa rivers. As a result of changing seasonal as well as daily weather conditions, the chlorine demand of the water after rapid sand filtration without coagulation and sedimentation may vary between 1.0 and 0.1 parts per million. The large variation depends upon the volume percentage of Ottawa water, with its high content of soluble organic matter, in the intake mixture.

It must be indicated that the soluble organic content of the Ottawa water as obtained is not comparable to that of the same water in, for example, the Lake of Two Mountains above Montreal Island. Water of Ottawa origin reaching the Montreal intake brings with it the domestic and industrial waste of several communities above it.

Colorimetric estimation of residual chlorine by means of light transmitted through liquid paths of definite length and intercepted by photo-sensitive cells seemed the best solution of the problem. The Redeal-Evans Chlorometer, which depends upon the electrochemical action of a platinum copper couple in presence of free chlorine, was studied but was unsuitable (1).

The first experimental work was described in the JOURNAL of May, 1931 (2). The selenium light sensitive cells available at that time, however, were unsuited for practical service.

A paper presented on March 27, 1940, at the Canadian Section Meeting at London, Ont., by J. H. Harrington, Chemist, Filtration Works, Montreal.

Early in 1931 a photo-electric bench using Western 1-A cells, was put in service controlling and recording the residual chlorine content of the treated water on plant scale. The apparatus was extremely sensitive but because of countless variables it was very hard to keep "in tune" and so was found unsuitable for serious development. This phase of the work has not been published except for an article by Hayes describing some of the electrical and mechanical features (3).

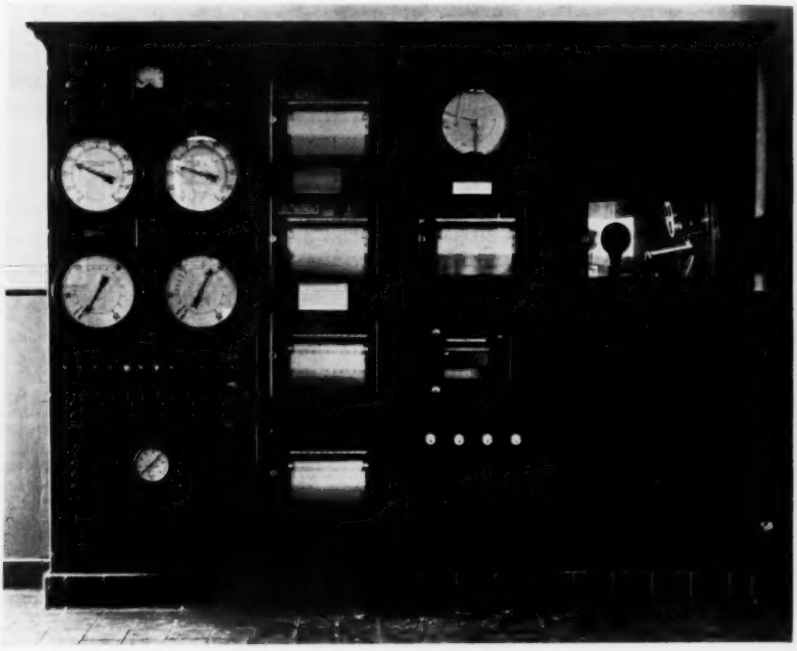

FIG. 1. Instrument Board

Soon after the barrier-layer type of photo-sensitive cells became available an experimental unit was set up on a laboratory scale and after several months observation of its performance an improved photo-electric bench was installed in April, 1935, which, with slight changes from time to time, has been controlling and recording residual chlorine content of the Montreal water supply since that time.

The combined filter effluents are chlorinated at the point of entrance to the filtered water reservoir and after passing through

this, the normal volume change being equivalent to a minimum retention period of two hours, the water, then sterile within the meaning of the public health standards for quality of drinking water, is delivered to the consumer services. The water entering the reservoir has an average residual chlorine content of about 0.05 parts per million, as of approximately ten minutes absorption. This residual chlorine is entirely absorbed during the retention period and none is present in the consumer services.

The chlorine dose is adjusted automatically to maintain a constant residual content after an absorption period of ten minutes and is fed

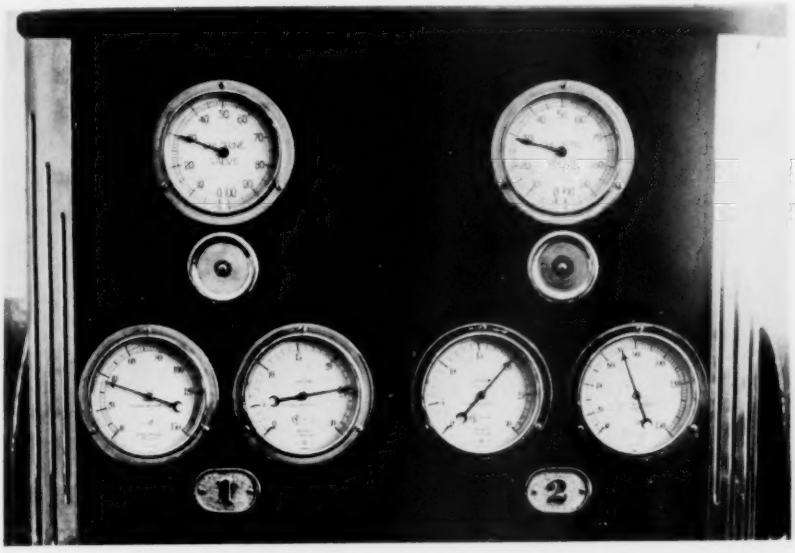

FIG. 2. Chlorinator Gage Panel

in accordance with the chlorine demand which varies with the rate of filtration and the biochemical demand. Rate of filtration varies from a minimum of 96 m.g.d. to a maximum of 168 m.g.d.; and biochemical demand varies seasonally and daily with wind changes. The seasonal variations are related to very cold or very dry weather and to the necessity of using the shore intake water having a high biochemical chlorine demand due to the high volume percentage of Ottawa river water and to sanitary pollution caused by the discharge of the sewage from fourteen towns within twenty-five miles above the intake. In cold weather the outer intake becomes choked

with anchor ice, and in a dry season its capacity is lessened because of the low river level. Either condition forces the use of more or less water of high pollution. Flood season may be coincident with an increase in chlorine demand but the concentration of sanitary pollution is low for obvious reasons.

A change of wind either in direction or velocity will cause a change in biochemical chlorine demand by altering the line of mergence of the St. Lawrence and Ottawa rivers with reference to the point of location of the outer intake, and consequently change the composition of the intake water. The two rivers meet at St. Anne de Bellevue

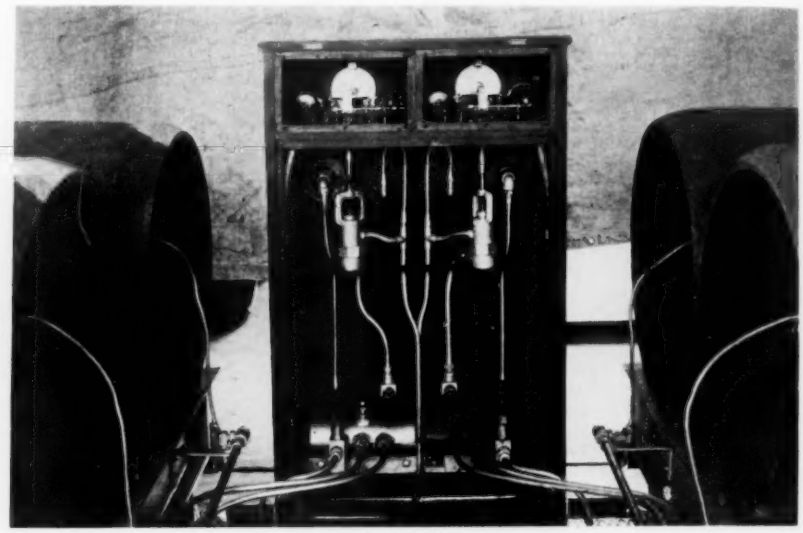

FIG. 3. Chlorinator Mechanism

about eighteen miles above the intake. The line of demarcation is plainly visible since the Ottawa water holds in solution a brown vegetable stain and presents a strong contrast to the parallel flowing St. Lawrence which reflects the familiar "blue" of nearly "colorless" water. Because of this content of organic matter in solution the chlorine demand of the Ottawa water is much higher. To insure a given residual chlorine content after a ten minute absorption period these two chemically divergent waters must be treated as shown in Table 1.

In recent years the problem has become increasingly complicated due to the conditions mentioned above.

The chlorine demand varying from any of the causes cited is satisfied automatically and continuously by means of the differential photo-electric bench.

The combined filter effluents are thoroughly mixed with the chlorine solution at Gate House 4 where a violent agitation takes place

TABLE 1

Treatment of Two Chemically Divergent Waters

RIVER WATER	VOLUME IN MIXTURE	ALKALINITY	REQUIRED CHLORINE PER MIL. GAL.	DATE
	per cent	*p.p.m.*	*lb.*	
St. Lawrence.....	100	107	0.5	Mar. 25, 1923
Ottawa.....	77	39	13.0	July 31, 1924

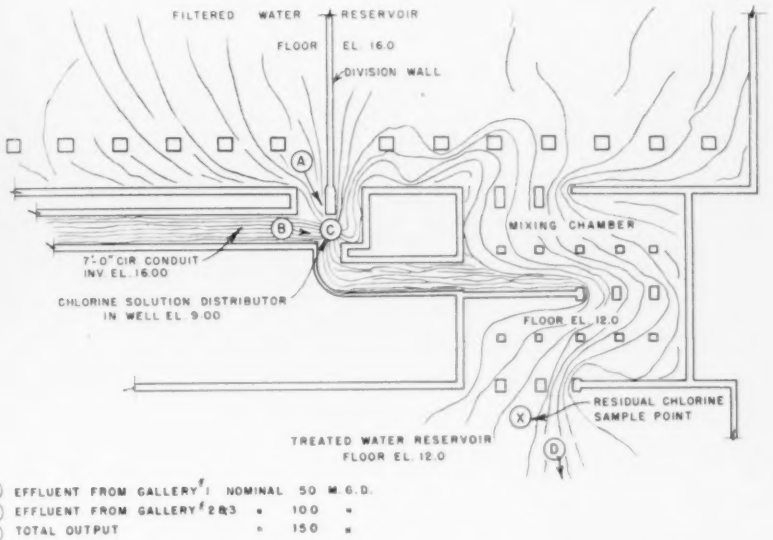

FIG. 4. Approximate Rates and Stream Lines of Filter Effluents at Chlorination Point (Montreal Filtration Works)

(see Fig. 4). The chlorine solution is applied to the water in the approximate volume ratio of 1/12,000. Following this, the treated water flow is baffled three times before entering the reservoir where the residual chlorine sample point is located, the chlorine solution and water being uniformly mixed by this time. A small pump draws a continuous sample of the treated water entering the clear well and

discharges it through 250 ft. of ½-inch copper pipe to the control board in Gallery 3 where the photo-electric apparatus is located. The total time between the application of the chlorine solution and the arrival of the sample in Gallery 3 is approximately ten minutes and is called the absorption period. Chlorine is absorbed quite rapidly after being mixed with the water and it has been found that a small residual amount remaining in the treated water after ten minutes can be taken as proof of sterility within the meaning of the public health standards for sanitary quality of potable water. Keep-

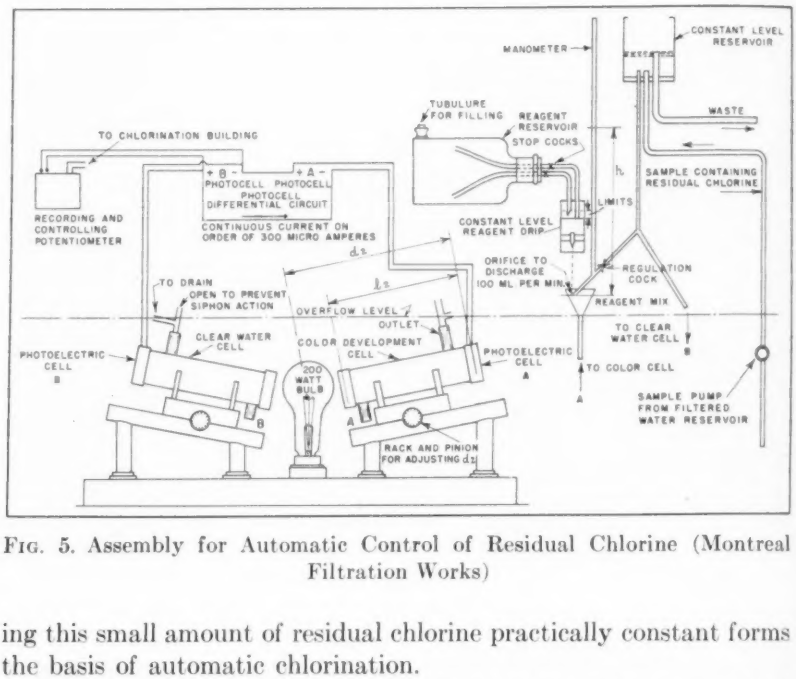

FIG. 5. Assembly for Automatic Control of Residual Chlorine (Montreal Filtration Works)

ing this small amount of residual chlorine practically constant forms the basis of automatic chlorination.

The sample containing the residual chlorine to be measured is delivered to a constant level orifice box above the photo-electric machine and from there two streams, of about 100 ml. per minute, flow respectively through each of two water cells which are diametrically opposite each other in the path of a common light source (see Fig. 5). The beams of light after passing through the water (10 in. liquid depth) are intercepted respectively by two photo-electric cells of the barrier-layer type connected in a differential circuit which is balanced electrically for clear water in both cells. Having obtained this electrical balance which is recorded as zero

on a potentiometer chart, a suitable reagent is continuously added to the sample entering only one of the water cells. This cell develops a color of an intensity strictly proportional to the amount of residual chlorine present in the sample, which color intensity in turn alters the characteristics of the light flux intercepted by the photo-electric cell on that side. The final result is the upset of the electrical balance by a definite amount proportional to the residual chlorine content of the sample, causing the potentiometer pen to move from zero, recording the content.

A standard recording and controlling potentiometer is used. The electrical potential, corresponding to the desired constant residual chlorine content, is chosen conveniently near the center of the

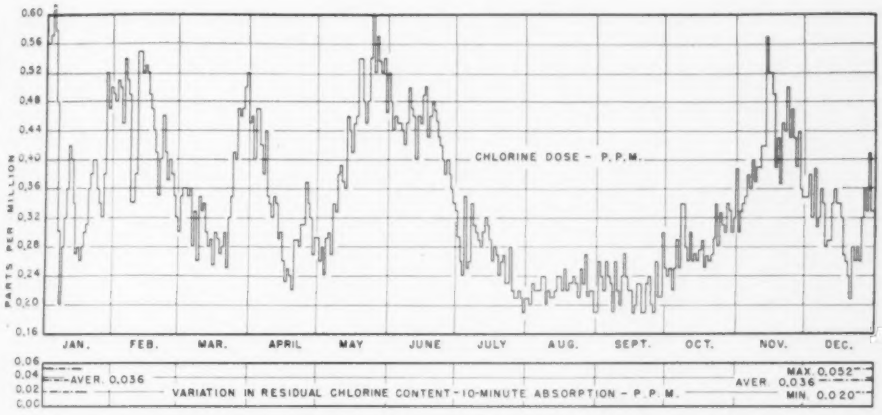

FIG. 6. Chlorine Demand and Residual for a Typical Year

chart. The method practiced is to set this at 25 mv. on an 11-inch, 0 to 50 mv. chart, with high and low limit contacts at 30 and 24 mv. respectively.

Obviously the residual chlorine content may vary between the above limits without any chlorine flow control valve movement. For residuals above and below these values the valve will close or open by a small increment for the purpose of adjusting the chlorine dose to meet variations in demand arising from any of the conditions already discussed.

The allowable movement of the pen in the neutral zone, between any given limits, is necessary to prevent movement of the chlorine flow control valve for very small residual content variations due to

lack of uniformity of primary distribution of the chlorine in the main volume of water at the dosage point.

Since the elapsed time between the primary addition of the chlorine in Gate House 4 and the measurement of residual in Gallery 3 is ten minutes, a timing apparatus is inserted in the chlorine valve motor circuit to prevent its being energized except at the ten minute intervals. Otherwise it would be impossible to regulate the residual content within close limits.

In practice, the present photo-electric bench maintains a practically constant residual. The graph of chlorine demand and residual for a typical year illustrates this very well (see Fig. 6).

Among the practical advantages to be realized from the use of automatic control of chlorine feed, two deserve special mention:

(1) The flow of the reagent and sample being continuous, the record of dosage is an integral one; an abnormal condition for the smallest fraction of a second would affect the record by the amount of its weighted average.

(2) The record showing the continuous maintenance of a practically constant residual chlorine content after ten minutes absorption may be accepted as proof that the water so treated is continuously sterile within the meaning of the public health standards for quality of water for domestic use. The truth of this statement is found in the correlation of tests for residual chlorine content and bacteria in samples taken for confirmatory bacteriological examination in the laboratory. Five 10 ml. portions collected at the residual sample point in Gallery 3 usually show no incidence of coliform group organisms. Twenty-two positives appeared in 4,635 tests made in 1939.

There are several barrier-layer types of photo-sensitive cells commercially available. Two of these have been studied in Montreal.

Figure 7 shows the relative spectral sensitivity curves of three types. It may be seen from these curves that cell C exhibits a relatively large change in spectral sensitivity for small changes in wave length. For the purpose of the study this indicated the choice of a reagent developing a color with residual chlorine lying in the blue transmission band.

There are at least three such reagents, starch-iodide, reduced crystal violet at pH 4.9–6.0, and ortho-tolidine at pH 4.4 to 5.5. Ortho-tolidine being easily obtainable has been the one in practical use.

In noting the required conditions for recording residual chlorine content of a given treated water, there are certain limitations which must be provided for. First, the mixture of the dosing solution with the water to be treated must be reasonably good. The efficiency of the mix is easily measured by taking a series of fifteen or more ortho-tolidine tests at intervals of five seconds, at the sample point. The volume need not be large; 10 ml. portions are convenient and sufficient to obtain comparative results. The information so obtained is very helpful in determining the capacity of the color cell of the photo-electric bench which should be designed so as to average

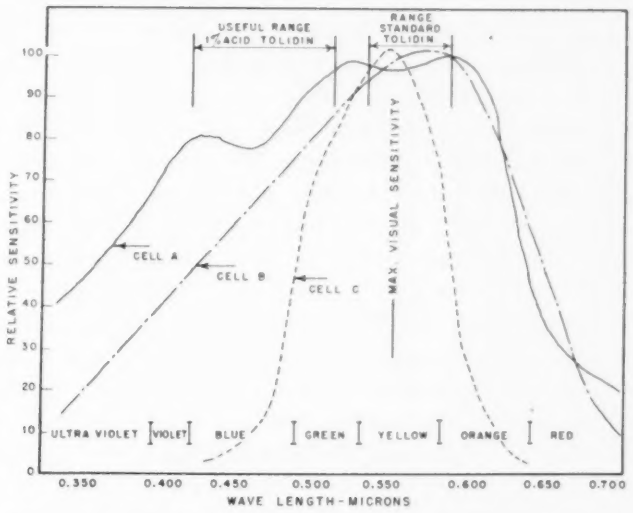

FIG. 7. Relative Spectral Sensitivity of Three Commercially Available Photo-electric Cells

a number of such tests, but limit the total time through the cell to not over three minutes.

Economical use of reagent and servicing of the apparatus is also important. Since a reagent reservoir of about two liters is a convenient size and should not have to be filled more often than once each day, the rate of reagent flow should be about 1 ml. per min. A rate of flow of approximately 100 ml. per min. is also convenient for the treated water in which the residual chlorine content is to be recorded.

Using 1 per cent acid ortho-tolidine solution as the reagent it has been found that for waters of the Ottawa and St. Lawrence this

volume ratio of 1 ml. reagent to 100 ml. of sample develops a color which falls closely within the useful range of the barrier-layer type photo-sensitive cells.

The conditions for optimum blue are easily checked by adding 100 ml. portions of the treated water, containing approximately the desired ten-minute residual chlorine content, to a series of tubes containing graduated volumes of ortho-tolidine solution of 1 per cent acid strength. For St. Lawrence water the optimum ratio is 1.5 ml. to 100 ml. and for Ottawa it is 0.7 ml. to 100 ml. In practice the optimum ratio is found by adjusting either the head on the reagent drip or that on the sample flow. The facts that the above series of tubes shows color passing through blue and green, and that the spectral sensitivity curves in this region are steep explain why this volume ratio of reagent to sample is not critical.

Given a suitable reagent, the sensitivity of the recorded measurement of a unit quantity of residual chlorine is a function of the light source, the length of the transmission path, and the range of the potentiometer. Using a 200-watt lamp a transmission path of ten inches and a potentiometer of 0 to 50 mv. range, a sensitivity of about 5.0 mv. for 0.01 p.p.m. residual chlorine is possible. This is too great for practical use and is reduced to workable limits by shunting a part of the galvanometer current through a suitable resistance. It has been found that a shunt resistance of 800 ohms gives a sensitivity of about 2 mv. for 0.01 p.p.m. residual chlorine which is ample both for recording and keeping a constant residual chlorine content.

Of six possible sources of error only two have been found worthy of consideration and of these only one is serious. The sources, in order of importance, are: (1) cumulative stain on the glass windows of the color development cell due to progressive decomposition of the reagent; (2) changes in light source line voltage; (3) changes in pH, i.e., conditions for optimum blue; (4) changes in velocity of color development due to presence of ammonia compounds; (5) accumulation of bubbles in interior of cells through liberation of dissolved oxygen; and (6) condensation of moisture on surface of windows of water cells.

Reagent stain must be compensated by movement of the color cell toward the light source by small increments as indicated by daily standardization.

Error due to fluctuating line voltage if serious will be disclosed in

the record of clear water balance obtained each time after washing out the water cells (see Fig. 8).

Source 3 is not critical as already explained.

During the spring break-up of the ice in the rivers, sufficient atmospheric ammonia compounds are present in the water to simulate

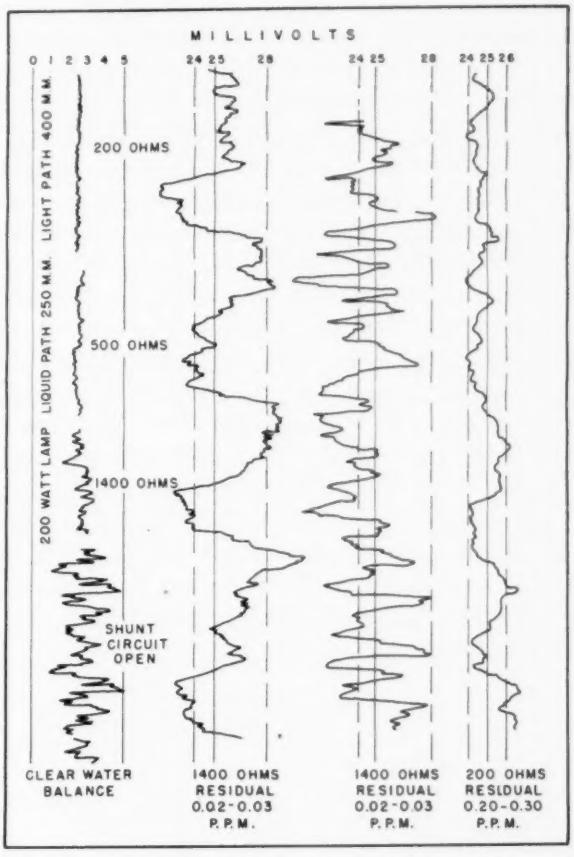

FIG. 8. Typical Forms of Recorded Residual Chlorine (Montreal Filtration Works)

chloramine treatment. The treated water sample after sensitizing with reagent must be given an extra ten-minute retention by leading it through a suitable length of one-inch hose before reaching the color development cell.

Errors due to air bubbles and moisture are not critical and in any case would be canceled out by the arrangement of the apparatus.

There has been no opportunity for studying the possible effect of the interference of nitrites and manganic manganese on color development with 1 per cent ortho-tolidin as used. It has been noted, however, that interference from these compounds does not occur at a pH above 4.0(4).

A dependable assembly of apparatus for recording the residual chlorine content of a water so sterilized, whether based upon photo-electric or electro-chemical phenomena, should have a wide application in the water purification field since it offers a means of proof, within the reach of the majority of water works officials, that the finished water is continuously sterile within the meaning of the accepted public health standards for sanitary quality.

Automatic control of residual chlorine content, on the other hand, may be more limited in its application due possibly to the fact that the general characteristics of a water supply being normal, at least for long periods, very little if any saving will be realized by applying such control to an inherently inexpensive treatment. But, given a dependable recording assembly for residual chlorine content, automatic control is but a short step further. It is indicated as indispensable for a supply however small where the characteristics of the raw water show sudden changes in organic pollution without warning.

In all other cases the installation for one motor-operated valve between the chlorine container and the ejector, which will simultaneously and automatically adjust the gas flow to variable tank pressure, water flow and biochemical chlorine demand, makes a strong appeal to the imagination.

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ABSTRACTS OF WATER WORKS LITERATURE

Key. **31:** 481 (Mar. '39) indicates volume 31, page 481, issue dated March 1939. If the publication is paged by issues, **31:** 3: 481 (Mar. '39) indicates volume 31, number 3, page 481. Material enclosed in starred brackets, ★[ ]★, is comment or opinion of abstractor. Initials following an abstract indicate reproduction, by permission, from periodicals as follows: *B. H.—Bulletin of Hygiene (British)*; *C. A.—Chemical Abstracts*; *P. H. E. A. Public Health Engineering Abstracts*; *W. P. R.—Water Pollution Research (British)*; *I. M.—Institute of Metals (British)*.

TREATMENT—GENERAL

Physical and Physical-Chemical Laws in Water Treatment. ERICH NAUMANN. Gas-u. Wasser. **82:** 353 (May 20, '39). In slow sand filters main action is biological and takes place in the Schmutzdecke, whereas in rapid sand filters adsorptive boundary actions are more effective than straining action. For this reason rapid sand filters are used in iron and manganese removal. Natural waters have individuality which is not always shown by chem. analysis, but only by special experimentation in method of treatment. Individuality affects sludge or floc formation, which has to be brought to desired physical condition. To get good results physical methods have to be used besides chem. reaction. Particular mention is made of contact beds containing coke, slag or lava, coagulation basins or pH adjustment. In filter washing removal, of dirt is obtained by friction between sand grains. Based on Stoke's law it is shown that more intensive collisions are obtained by use of slow water velocities combined with compressed air than by a high water velocity alone. As filter depends on surface action, nature of grain surface is important. If surface is inert, as with quartz, we have a physical filter, but if surface can also be chemically active, as with marble, Magno-mass, or zeolite, we have chemical filters, although in these physical action is also effective. The adsorptive forces are of an electrical nature due to unsaturated valences of uppermost layers of atoms or molecules. Molecules with dipole moments and electrically charged colloids may show differences in adsorption. Treatments for boiler waters are mainly of a chem. nature, but use of physical processes has been started with tonisator and with high frequency currents. Processes may some day show success. Shown that adsorption coefficient of water for air and oxygen, viscosity, specific volume, dielectrical constant, specific heat, electrical dissociation and the refractical index of water show a breaking point between 60°C. and 70°C., functions being lineal above this temp. This

is due to depolymerization in molecular composition of water at this temp. and may explain the increase in aggressive properties of the water, especially increased effect of dissolved oxygen, as well as killing effect on most pathogenic organisms.—*Max Suter.*

One Chemical, Three Treatments at El Segundo Water Works. LOUIS J. ALEXANDER. Am. City **54:** 9: 51 (Sept. '39). Water for El Segundo, Calif. is derived from subterranean peat beds and deeper-lying petroleum measures and consequently supports heavy growths of slime-forming bacteria which produce repulsive odors and tastes. Several types of treatment failed and another study was initiated. Decided to use as simple a treatment process as possible to avoid need for employing highly technical operators. 16 grains of hydrated lime per gal. found sufficient to raise the pH to 10.5 at which point bacterial growth was inhibited. Lime floc produced by softening reaction removed much of organic matter while total hardness was reduced to 150 p.p.m. Successful operation of plant on this schedule has justified its use. New plant built embodying following processes: collection of raw water from wells, aeration, mixing, flocculation, sedimentation, filtration and transfer of filtered water to main storage reservoir. Water dept. of El Segundo is self supporting and this new construction was carried out on a force-account basis and without receiving any outside financial assistance. Treatment units are outside while mechanical equipment, chem. storage, lab. and offices are in 3-story operating building.—*Arthur P. Miller.*

Flexible Treatment of Water Checks Tastes and Odors. ANON. Am. City **54:** 3: 46 (Mar. '39). Filtration plant at Chicopee, Mass., designed for 5 m.g.d. regularly delivers 5.6 m.g.d. Water is secured from Cooley Brook Res. with capac. of 150 mil. gal. This res., created by an earth dam with concrete core wall resting on a 53′ steel sheet piling cutoff wall, is supplemented by the old Cooley Brook Res. of 3 mil. gal. and Morton Brook Res., latter used only in emergency. New Cooley Brook Res. is surrounded by city-owned watershed which is carefully tended. From dam, water proceeds to battery of 36 "Spraco" nozzles on roof of sedimentation basin which reduces CO_2 from 5 to 2 p.p.m. and reduces odors. 2 mixing chambers which can be operated in series or separately receive aerated water to which alum has been applied and permit 15 min. mixing. Following this, 2 hr. sedimentation takes place and then filtration through 6 filters. During part of year lime is added to mixing chamber effluent to reduce CO_2 and increase alkalinity. Otherwise lime is added as water enters clear well. Filter runs ave. 7 to 8 hrs. Finished water is chlorinated and activated carbon regularly used at rates from 5 to 15 lb. per mil. gal. If and when pre-chlorination is necessary, it takes place at influent of mixing chamber and at that time carbon would be applied to effluent in mixing chamber. Plant designed so chem. application is very flexible. A large and well equipped lab. aids in control of water quality.—*Arthur P. Miller.*

Treating a Difficult Water in North Central Massachusetts. ANON. Am. City **54:** 2: 71 (Feb. '39). Raw water used at Athol, Mass. has ave. alkalinity

of 6, color from 30 to 180, and continuous growths of objectionable plankton. In spite of these conditions an odor- and taste-free water is produced. Only 2 of 3 impounding res. are used and water in equal quantities is taken from them and delivered to an aerator chamber. Lime is applied below "Aer-O-Mix" and alum at top, both of these chemicals being fed through dry feeders as well as soda ash which is applied to clear well. After aeration water flows to 2 settling tanks which are operated in parallel and give a detention period of 4 hr. Two ¾ m.g.d. filters complete treatment. Black alum is used continuously and has been very effective. Operating gages, storage facilities and pumping units are briefly described. A lab. equipped for standard chem. and physical tests is included. To combat odors, res. are treated with copper sulfate regularly during the periods of max. growth of plankton. Ave. use of water at Athol is ½ m.g.d. with pop. of 12,000. Most manufacturers use river water without treatment for their plants and city water for drinking purposes. —*Arthur P. Miller.*

Improvements Made During the Last Two Years at the Palermo Water Works of Buenos Aires (Argentine Republic). J. BACIGALUPI. Bol. Obras Sanitarias Nacion (Buenos Aires) **3:** 31 ('38). Account of recent alterations made to improve efficiency of Palermo Water Works at Buenos Aires, Argentine Republic. Coarse screens in intake on La Plata river have been replaced by coarse and fine screens; automatic metering of raw water has been introduced and pumping capacity has been increased; better coagulation has resulted since coagulant has been added to pump inlets instead of to settling tanks. Conditioning of floc by aeration has resulted in better coagulation, more rapid settlement, removal of aggressive carbon dioxide, and a 20% saving in coagulant. Introduction of a series of vertical columns at inlet ends of settling tanks has improved efficiency of sedimentation; filters are washed with water containing 1 p.p.m. hypochlorite, and water from settling tanks is chlorinated before filtration, thus diminishing amount of chlorine to be added to water before distribution and ensuring that filters remain free from bacteria. Storage tanks for filtered water have been reconstructed to improve facilities for cleaning; pumping plant and system of pipes at plant have also undergone alterations. Rapid sand filters are cleaned by compressed air and backwashing. Application of compressed air has disturbed the layers of sand and gravel in filters, and experiments are being carried out on design of filters to be washed with water only.—*W. P. R.*

Significant Experiences in the Treatment of Water in New York State. C. R. Cox. J. N. E. W. W. A. **53:** 444 (Dec. '39). Water treatment experiences in 3 N. Y. State communities recited, these showing effects of inadequate equipment in 2 instances. At Delhi, N. Y. (pop. 1800) ozone treatment applied from '28 to '35; water supply an upland lake water. Plant built had 0.75 m.g.d. capac., consisted of 60,000 gal. steel coagulation basin, 2 horizontal pressure filters, storage tank of 132,000 gal. capac., built at cost of $40,000. Ozonating equipment supposed to have capac. of 86 grams ozone per hr., equivalent to about 0.7 p.p.m. for flow of 0.75 m.g.d.; 60 grams of ozone were supposed to be produced per kw. hr. of electricity at 2¢ per kw. hr. Dose of

0.5 p.p.m. held adequate but proven to be incorrect. Difficulties encountered were high consumption of water due to leaks in distribution system, difficulty of ordinary operator to understand and operate equipment properly, and low solubility of ozone. When operating satisfactorily disinfection was secured without taste, these compounds being oxidized; some color was also removed. High cost of equipment a considerable obstacle. Later replaced by chlorination. Neither starch iodide nor ortho-tolidine test quantitatively correct for control of treatment, found difficult to change ozonation to meet demands of changes in organic matter present in water. Itemized cost data lacking. To meet the needs for an economical purification method at South Fallsburg, N. Y., where an organic and algae laden polluted surface-water was used, a compromise plant installed consisting of heavy chlorination, coagulation, filtration through 2 pressure filters at 3 gal. per min. and dechlorination in granular carbon filter at 6 gal. per sq. ft. Faulty operation, lack of mixing and settling and excessive rates resulted in failure to remove continuously all of suspended matter and tastes and odors; good bacterial results, however, obtained with no chlorinous tastes; method provided for effective treatment without close supervision; carbon removed chlorine long after ability to remove tastes and odors gone. This supply and partially successful treatment plant to be supplanted by well supply. Public supply at Elba, N. Y., derived from 2 wells in gypsum rock, water obtained has hardness of 1100 to 1500 p.p.m., softened by zeolite unit. One well equipped with deep-well pump other with air lift. When deep-well pump used, Fe in water remained in ferrous state and was completely removed by zeolite; when air lift well used, Fe was oxidized and zeolite clogged up giving poor softening results. H_2S odor due to biochem. reduction of sulfates to sulfides noticeable when deep-well pumped water used. Corrected by installation of coke tray aerator beyond zeolite unit; corrosiveness of water corrected through addition of 6 p.p.m. caustic soda and 6 p.p.m. sodium silicate. These experiences indicate newly developed equipment should be given thorough testing before selling to public; inadequate water purification equipment of little use; unusual waters require adequate lab. investigations to provide basis for design of plants for treatment.—*Martin E. Flentje.*

The Water Treatment for the Indoor Swimming Pool of the Empire Sporting Field. E. F. Dreher. Gesundheits-Ing. **62:** 429 (Aug. 5, '39). Pool is 82′ x 50′ and holds 420,000 gal. Water passed through a filter of 780 sq. ft. and recirculated by 885 g.p.m. pump. Number of visitors limited by requirements of clearness and hygienic properties of water. Found to be: turnover in gal. per day divided by 1,000, or for this pool 1,200 persons per day. Daily makeup water is 4% of basin contents. Filters were originally quartz sand filters and required alum as a flocculent to be effective. This lowered pH of water which was dangerous as it becomes aggressive and unhealthy for bathers and favors loss of chlorine into air. To keep water alkaline top 1′ of sand was replaced by Magno-mass which caused water to stay at a pH of 8. This required that sodium aluminate had to replace alum as latter does not flocculate easily at this high pH. Amount of aluminate necessary varies with no. of bathers. With 400 persons using the pool per day about 0.5 p.p.m. are needed, with 800

bathers per day, 0.75 p.p.m., and with 1,200 persons per day, 1 p.p.m. Chlorination is regulated to have in inflow 0.3 p.p.m. and in outflow a residual of 0.1 p.p.m. This requires addition of 0.5 p.p.m. Cl^- if 400 bathers use pool daily, 0.75 p.p.m. with 800 bathers and 1.0 p.p.m. with 1,200 bathers per day. Bacteriological tests show that this procedure is satisfactory. *Ibid.* p. 432. W. HAASE. Gives chem. reasoning for processes selected. Acidification produced by alum caused steady increase in odors of chlorine and considerable corrosion in iron pipes. Decrease in carbonate hardness also found. Addition of soda failed to correct these conditions. Use of Magno-mass and sodium aluminate gave stable alkaline water. Formation of magnesium hypochlorite from residual chlorine and Magno-mass prevented any chlorine odors even if 2 to 3 p.p.m. of Cl^- were present.—*Max Suter.*

Treatment of Water from Impounding Reservoirs, Lakes and Ponds. O. WASSER. L'Eau (Fr.) **31:** 135 (Dec. '38). Self-purification of streams involves combination of physical, chemical and biological factors and is upset by suddenly increased organic pollution. In impounding reservoirs, during high water periods water has characteristics of contaminated river water, since self-purification is slow process, and water is constantly being renewed. Water treatment must therefore be regulated accordingly. After water ceases to flow through spillway, self-purification proceeds on remaining water, but not rapidly enough, and putrefaction of organic matter leads to color, taste and algae troubles. Low pH resulting from putrefaction causes water to attack dam materials such as granites and ferruginous schists, thus releasing iron, and also hastens pipe corrosion. Plankton assisting in self-purification must also be removed in treatment process, as well as pseudo-plankton, such as Ascaridae. Treatment of these waters includes aeration and elimination of putrescible organic material by coagulation, assisted by prechlorination which also helps destroy plankton. Plankton are also retained on filters. Neutralization is effected by lime or by filtration through neutralizing material such as calcined dolomite. Filtration and sterilization should conclude treatment process. Activated carbon may be needed for taste and odor control. Above treatment may be carried out in pressure or gravity system, and diagram of each is shown. Each water must be studied individually. In pressure system, aeration precedes coagulation and also follows sand filtration. Contrary to general opinion, near saturation with air helps prevent corrosion by precipitating dissolved iron as a pipe coating of insoluble ferric compounds. Increase of pH and final chlorination terminate treatment. Gravity plants are very similar to those used for river water, varying with the designer. Principles are same as for pressure treatment but proper coagulation is of greater importance since it is not assisted by aeration. Remineralization of a very soft water may be desirable where simple neutralization does not repress aggressivity sufficiently.—*Selma Gottlieb.*

Water Purification—Current Tendencies. G. BAXTER. Tech. San Mun. (Fr.) **35:** 10, (Jan.-Feb. '40). During last 25 yr. bacteriological quality of water has greatly improved, due to more rigorous requirements. Insufficient

attention is yet paid to physical conditions. Storage important, but pollution factor of res. should not be neglected. Slow sand filters are unused about 35 days per year; coagulation, settling and rapid filtration permits high efficiencies. Sterilization by chlorine and chloramines is effective, but sometimes objected to because it may cause a disagreeable taste, destroys the properties of "natural" water, may be deleterious to health, may give false security when water is polluted. Objections may be answered. Active carbon will remove the taste, changes of natural conditions are more imaginary than real; nobody has shown that traces of chlorine are detrimental to health; failure to chlorinate is a greater risk. Danger of contamination exists when no pollution enters, caused by water plant personnel. Their periodic examination should be required; acquaint them with danger of typhoid pollution; new pipes and conduits should be sterilized; provide filter plant operators with proper tools, conveniences and flexible plants; draw up strict regulations for hygenic habits of personnel; eliminate air valves. About 900,000 cu. m. water is ozonated in France. When cost is of no importance ozonation is preferable to chlorination for sterilization, because it sterilizes and eliminates odors, does not react chemically with salts, destroys part of color, produces a pleasant taste. Modern technic of water purification permits transformation of polluted river water to potable water of good physical and bacteriological quality.—*Willem Rudolfs.*

Water Purification Progress During 1939. NORMAN J. HOWARD. Eng. Cont. Rec. **52:** 52: 49 (Dec. 27, '39). Federal anti-pollution activities and occurrence of water-borne disease in cities where source of supply is highly polluted have focused attention on water pollution. Increasing demand for more severe standards of bacterial quality for finished waters and has been suggested that such waters be examined for coliform bacteria in 100 ml. portions. Committee on Standard Methods of Analysis, Canadian Public Health Assn., recommended standard lactose broth as primary medium for coliform bacteria test with direct confirmation in brilliant green bile broth. Much interest displayed in accelerated methods of treatment, which, while used mainly for softening, are said to be equally suitable for clarification of turbid or colored water. Porous plates successfully substituted for usual gravel layers for supporting filter sand at Larchmount, N. Y., additional freeboard thus obtained, permitting higher rates of backwash. Superchlorination gaining favor for taste elimination owing to additional protection afforded. Contact period important factor in this treatment. Taste usually intensified as chlorine dosage is increased until oxidation dose is reached. Faber's researches have thrown much light on superchlorination. Goiter has been associated with water supply for many years, chiefly from viewpoint of iodine efficiency. Andre Crotti, in Switzerland, contends goiter is specifically water-borne and is caused by "goiter fungi," which have also been isolated from raw vegetables. Fungi are not easily removed by sand filtration but readily destroyed by heat. Red brass pipe containing 85% copper has been found more resistant to corrosion than copper tubing but brasses containing lower percentages of copper not satisfactory.—*R. E. Thompson.*

Construction of a Model of a Purification Plant. ROBERT S. MILLAR. Pub. Wks. **70:** 2: 11 (Feb. '39). Model of plant comprising sedimentation, coagulation, filtration and clear water storage is described. Made of glass with $\frac{3}{4}'' \times \frac{3}{4}''$ angle iron frames, glass plates sealed together with aquarium cement; basins $14\frac{1}{2}'' \times 20'' \times 12\frac{1}{2}''$ deep. Chem. solutions fed from bottles and water with turbidity of 1000 used for demonstrations. Filter contains 6″ sand, $2\frac{1}{2}''$ gravel with $\frac{1}{4}''$ underdrains of copper.—*Martin E. Flentje.*

Experiences and Observations. F. L. COVENTRY. W. W. and Sew. **86:** 484 (Dec. '39). Filtered water contained high organic matter, causing abnormal chlorine disappearance. Prechlorination solved this difficulty. For corrosion control, steel-wool contact chamber is added to Enslow stability indicator, followed by white porcelain drip plate to detect undersaturation. Recarbonation of lime-soda softened water with CO_2-bearing raw water works successfully. In one place, both impounded and well supplies are available: one high in non-carbonate hardness; other containing 96 p.p.m. Na_2CO_3. A 50-50 mix, with lime added, produces excellent effluent. Inclusion of hypochlorite in jute used for joint calking improves sterilization of new mains.—*H. E. Hudson, Jr.*

The Graphic Representation of the Acidity of Soft Waters. JEAN FRANQUIN. Tech. Sanit. Munic. (Fr.) **34:** 40 (Apr. '39). Diagrams show how water may be classified according to the interrelation of its 3 values: pH, semi-combined CO_2 and free CO_2. Diagrams make it possible to gage composition that a water should possess to arrive at an equilibrium, a point where it may be susceptible to deposit a protective coating on pipes. Shows the pH this treated water should possess. Assumed 3 kinds of treatment: marble or similar synthetic products, aeration, lime. Where combined CO_2 is <25 p.p.m. ordinary aeration alone is insufficient to bring water to equilibrium. Actually, aeration does not completely eliminate the free CO_2, and consequently, straight line curve cannot be attained. In waters where total semi-combined CO_2 plus free CO_2 is < 25 p.p.m., there is no danger of raising the pH of treated water too high. May attain values between 8.3 and 9.9 according to free CO_2 content. In this case all free CO_2 will be aggressive. In waters where total of semi-combined CO_2 plus free CO_2 is >25 p.p.m., free CO_2 decomposes to: CO_2 aggressive to marble and equilibrating CO_2. Free CO_2 is shown by diagram.—*Willem Rudolfs.*

Water Pollution Research. Extract from Annual Report of the Water Pollution Research Board. ANON. Wtr. and Wtr. Eng. (Br.) **42:** 46 (Feb. '40). Public water supplies threatened by pollution must be adequately treated. Board has conducted research on water treatment, sewage disposal, and stream pollution. Examinations of British base exchange materials have been made. Within the limits of 4° to 20°C. temp. has no effect on base exchange materials tested. Passing of water through limestone or marble reduces corrosive action on lead. Research lab. set up to investigate factors which govern abundance, types, and biochem. activity of bacteria in fresh water. Preliminary investigations indicate correlation between rainfall and

numbers of bacteria in upper strata of water in Lake Windemere. Sodium caseinate agar was found to be most suitable medium tried. Colonies are counted after incubation for 15 days at 20°C.—*H. E. Babbitt.*

Removal of Silica from Water by Sodium Aluminate. F. K. Lindsay and J. W. Ryznar. Ind. Eng. Chem. **31:** 859 (Jul. '39). Describes expts. on removal of silica from water by treatment with sodium aluminate and with Ferrisul (commercial ferric sulfate). Amt. of silica removed from water by hydrated aluminium oxide proportional to concentration of silica, i.e., adsorption takes place and F. l.c. isotherm is applicable. Max. amt. of adsorption occurs between the pH values 8.3 and 8.7; optimum pH value for removal of silica by ferric sulfate is about 9.0. Found best results were obtained when aluminate and sufficient lime to give optimum pH value were added to water, after being mixed together. When more than about 34 p.p.m. coagulant was added, sodium aluminate gave greater reductions in silica than ferric sulfate. After adding sodium aluminate in a concentration of 255 p.p.m. to a sample of water containing silica, settled sludge was separated and added with 34 p.p.m. aluminate to another sample of water; sludge from second precipitation, with 34 p.p.m. aluminate, could be re-used, and so on. In 8 such cycles, average amount of silica left in treated water was 0.7 p.p.m. when initial concentration of silica was 3.3 p.p.m., and 3.39 p.p.m. when untreated water contained 16.6 p.p.m. silica; corresponding figures for residual concentration of silica when aluminate was replaced for the same amt. of ferric sulfate were 1.56 p.p.m. and 8.47 p.p.m. Both coagulants gave better results when added with sludge than when used alone.—*W. P. R.*

Sanitary Engineering. Edward J. Cleary. Eng. News-Rec. **124:** 226 (Feb. 15, '40). Construction of water supply and sewage disposal facilities reached all-time high in '39. Of chief interest in water purification were developments in ozonation, superchlorination, and treatment with sodium hexametaphosphate for inhibiting corrosion and preventing calcium carbonate scale formation. Two ozone plants are under construction as result of improved equipment, which has reduced operating costs. Superchlorination has been shown to be effective for taste control at no. of plants, dosages as high as 7 p.p.m. being employed. Cathodic protection has been demonstrated to be effective means of controlling corrosion of steel water tanks. In distr. field, new design of c.i. pipe, emphasizing importance of external loading, was announced, and importance studies of water hammer in long pipelines were made in Los Angeles. Studies of life of c.i. pipe have indicated ave. life of 150–170 yrs. for 8″ pipe and longer for larger pipe. For water storage, dual purpose structures and spheroidal tanks made their appearance.—*R. E. Thompson.*

The Ideas of a Small Plant Operator. A. E. Clement. Penna. W. W. Opera. Assn. **10:** 76 ('38). Water purification plant at Natrona, Penna., has capac. of 1.0 m.g.d. Source of supply is Allegheny R., water of which ranges in pH from 3.0 to 7.0 and turbidity of 10 to 2,000. Location of rains affects pH, which can run entire range in a few hours; and rains and stage of the river

affect turbidity. Raw water is chemically treated with alum applied at pipe riser just before entering mixing chamber, followed by lime when turbidity is 100 or less, by sodium aluminate when turbidities are over 200 or at direction of operator, and by activated carbon. Quantities vary according to pH for alum and lime. About 30 p.p.m. of alum are required at pH 6.6. Sodium aluminate is added for excellent floc-forming qualities in doses of 10 p.p.m. or less at discretion of operator and pure 98% lime in amounts of about 15 p.p.m. when pH is 6.6 and 50 p.p.m. of hardness. Activated carbon is applied at mixing baffles in doses varying from 0 to 10 p.p.m. All water samples are piped from various control points directly to lab. in filter plant. Readings of pH are taken and recorded every hour, as are residual chlorine test, settings of chem. feed machines, and floc conditions. Methyl orange test and turbidity are taken at least once during 8-hour shift. Each morning a part-time chemist checks determinations made by the operator.—*P. H. E. A.*

CHEMICAL FEEDING, CONDITIONING, SEDIMENTATION

Symposium on Lime. Amer. Soc. of Testing Materials (1940). *Factors Affecting Milk of Lime Employed in Industry.* D. E. WASHBURN. p. 36. Lime used for diverse industrial processes—caustization, coagulation, distillation, precipitation, neutralization, etc. Quicklimes used contain a min. of 90% CaO and < 3% $CaCO_3$. Reason exists for believing that in reactions involving CaO, high suspensions and increased vol. of sludge tend towards colloidal states that are aid to chem. reactivity. Investigation made to throw some light on effect of hard burning in the mfr. of high-Ca quicklimes as a factor in governing suspension values and final vol. of solids in milks of lime. Shaft and rotary kilns used in mfr. of CaO, temp. employed 2000° to 2500° F., exposure time of stone to heat 6 hr. in shaft kiln, and not over 1 hr. in rotary. Stone 5″ to 12″ charged to shaft kiln, rotary feed has max. size charge of 1½″. In study of 6 quicklimes and 1 hydrated lime it was found that harder a stone was burned less was vol. of solids. No direct correlation found for settling rate; samples 1 and 2, rotary kiln lime of porous stone and hard burned, settled slowly, porosity being an evident factor; sample 3, a shaft kiln lime of dense stone with mild burning also settled slowly while samples 4 and 5, of same stone but with hard degree of burning, settled rapidly; sample 6 was of same stone as 3, 4 and 5 but was calcined in a rotary kiln, mild burning for 1 hr. gave slowest rate of settling. Author believes slow settling is due to low density particles, governed by slaking rates. Hard burning reduces slaking rate (by reducing porosity), and ready penetration of water to induce rapid slaking. Rapid slaking has bombarding effect on particles to reduce densities. *Quality of Lime Desired for Water Treatment.* CHARLES P. HOOVER. p. 43. Lime widely used in water treatment. Among uses, softening for removal of Mg and carbonate hardness, as coagulant in hard waters, adjustment of pH before and after filtration. Either quicklime or hydrated lime of high Ca content used. Hydrated lime more generally used in smaller plants, keeps better in storage and does not require slaking equipment. If quicklime to be used, kind and size used depends on feeding devices and time available for slaking. At Columbus, Ohio found additional cost of handling lime that would not slake in cold water was 20¢ per ton. In specifications

bonus and penalty clause should be included based on CaO content of local stones. Ellm's and sugar tests for water-soluble CaO given. In discussion effect of F in lime brought up. Author gives F content of 7 limes (analyses by Ohio Health Dept.), varying from min. of 52 to max. of 176 mg. F per kg. lime; with lime dosage of 20 grains lime per gallon and F content of 100 mg. per kg. F applied to water would only be 0.034 p.p.m. *Prevention of After-Precipitation in Lime-Treated Industrial and Municipal Waters by Threshold Treatment.* Owen Rice, Everett P. Partridge and R. E. Hall. p. 50. Threshold treatment of calcium carbonate waters consisting of addition of 1 to 2 p.p.m. of sodium hexametaphosphate preventing pptn. of $CaCO_3$ explained by this chemical prevention or killing off of crystal nuclei in such solns. as rapidly as formed, leaving the soln. satd. with $CaCO_3$ in soln. One application of property found in hot-process lime-soda softeners where application kept tanks and lines clean without interfering with softening reaction; quantity required even in large softener not over 1 lb. per day. Continuous application of 1 p.p.m. in effluent of cold softeners will prevent after-pptn. of $CaCO_3$ on valves, lines, etc. May also be used with lime for prevention of corrosion, action persists when water heated to 80°C. or higher; chemical non-toxic even when ingested in amts. for surpassing water supply use. *Soil Acidity and Liming, and Some Factors that Should Determine the Standards for Lime for the Soil.* Elmer O. Fippin. p. 56. In '37 use of liming material ranged from 0.9 lb. per acre in Miss. to approx. 100 lb. in Ky., present total use 6.8 mil. tons. Dominant mineral colloid in soil is silicon, which in SiO_2 form with water forms silicic acid and is built up into numerous complex molecular aggregates with varying degrees of acidity and basicity. Functions of liming materials in soil: (1) to regulate pH balance, (2) to supply and regulate plant nutrients, (3) to suppress effect of toxic agents, (4) to increase availability of essential nutrients, (5) to control disease conditions, and (6) to improve condition of soil. Calcium carbonate or lime may be used, dependent on cost and convenience. *Preliminary Investigation of the Determination of Fluorine in Lime.* C. J. Koehler. p. 83. Because F is found in lime (in conc. from few to 50 p.p.m.) and is used in preparation of water and foods, desirable to have method for detn. of this element. Test made on 20 grains of lime placed in Claissen-type distilling flask connected through a separatory funnel to a steam generator (glass flask) and through outlet tube to spiral condenser attached vertically. 20 grams lime, 30 ml. water and 20 grams glass beads placed in flask, 100 ml. 60% perchloric acid added, contents heated to 100°C. and steam generator started, temp. then adjusted to 150 ±1°C. 80 ml. fractions taken, 80 ml. alcohol added to fraction with 4 drops sodium alizarine-sulfonate indicator (0.1 gram in 100 ml. H_2O filtered). Fraction made alkaline with 0.1 *N* NaOH then just acid with 0.1 *N* HCl and 2 ml. buffer soln. added (9.448 grams monochloracetic acid and 2 grams NaOH in 100 ml. H_2O); soln. then titrated with thorium nitrate soln. (strength such that 1 ml. equiv. to 0.0002 gram F, standardized against NaF soln.). Found relatively large quantity of distillate needed to catch all of fluorine; data when treated mathematically by formula: $\frac{dx}{dy} = ax$; with dx amt. F carried over in distillate dy with a a constant. Does not agree with theory exactly,

although giving fair agreement. To be studied further. *Fundamental Mechanics of Calcination and Hydration of Lime and Methods of Control.* VICTOR J. AZBE. One of marvels of calcination is that limestone calcined at mildest temp. for shortest necessary time retains as lime original dimensions of stone even though it has lost 44% of original wt., and with true sp. gr. of 3.4 has apparent gravity of only 1.46. Half of vol. is free space. Hydration a surface reaction. Water drawn in with great force, reaction proceeding along myriads of surfaces. Impairment of lime structure greatly decreases rates of reaction. Grinding may also have effect on reaction rates. Considerable discussion given to methods of lime plant operation.—*Martin E. Flentje.*

Lime for Water Works Use. J. G. THOMPSON. J. N. E. W. W. A. **53:** 255 (Sep. '39). Mfgrs. problem in making lime is resolved to essentials of having a lime deposit from which to recover lime of high calcium carbonate content, necessary plant layout consisting of kilns, mfg., loading, and packing equipment and proper personnel. To produce quick lime of 90% CaO and hydrated lime of 68.1% CaO content, quarry material must have 95% $CaCO_3$. Either rotary or vertical shaft kilns used. In shaft kilns great deal depends upon sorters and pickers. Flow sheet given.—*Martin E. Flentje.*

Manufacture of Alum, Ferric Sulfate and Sodium Aluminate. F. A. ABBIATI. J. N. E. W. W. A. **53:** 247 (Sep. '39). While aluminum sulfate is of primary importance to water works field, author believes ferric sulfate and sodium aluminate will, in future, be more widely used. Alum made from bauxite found in French, Dutch and British Guiana, Yugoslavia and in U. S. (Ark. and Ga. deposits used now). Can also be made from lower grade clays. Bauxite contains from 55 to 60% Al_2O_3, some Fe_2O_3 and titanium oxide, silica and moisture. Finely ground bauxite is digested in sulfuric acid, settled, washed and concentrated to solid alum. May be done in batch or continuous process. Sodium aluminate made in similar manner except caustic soda used in place of acid; bauxite must, however, have upper limit of approx. 5% SiO_2 to prevent loss of acid and Al_2O_3. Ferrisul, anhydrous ferric sulfate, made from pyrites cinder resulting from sulfuric acid production and sulfuric acid. Flow sheet for 3 processes given.—*Martin E. Flentje.*

Studies on the Storage of Ferrous Sulfate. WARREN A. KRAMER AND AUGUST V. GRAFF. W. W. Eng. **92:** 1275 (Sep. 27, '39). Commercial ferrous sulfate contains 98% to 99% $FeSO_4 \cdot 7H_2O$, 0.4–0.8% insoluble matter, 5 to 6 oz. per ton free acid and 0.1–0.2% ferric Fe. Nothing has been found equal to excess lime and ferrous sulfate for treatment of highly turbid water. Caking of material into lumps often takes place during storage. Results of tests show 2 possibilities available for satisfactorily storing ferrous sulfate: (1) control of temp. and humidity in bins within stability limits for $FeSO_4 \cdot 7H_2O$; (2) production of a product stable at ordinary temp. and humidity. Found commercially practical to produce material containing approx. 105% $FeSO_4 \cdot 7H_2O$ which gives little trouble in storage; if stored for longer than 4 to 6 weeks, $FeSO_4 \cdot 7H_2O$ content higher than 105% advisable.—*Martin E. Flentje.*

Photoelectric and Conductometric Method of Controlling the Addition of Aluminium Sulfate to Water. G. I. VAINSHTEIN. Zavodskaya Lab. **8:** 167 ('39). In adding $Al_2(SO_4)_3$ to river water dosage of coagulant is controlled by measuring light penetration of treated water with photoelec. cell. Galvanometer readings are compared every 2–4 hrs. with those obtained with untreated water. During periods of inundation, differences between treated and untreated water cannot be recorded with any accuracy because of high turbidity of untreated water. In such cases dosage of coagulant may be regulated by comparing elec. cond. of untreated and treated water. These methods may be used for continuous control of coagulant dosage.—*C. A.*

Copper Sulfate for Water Works Use. E. FITZPATRICK. J. N. E. W. W. A. **53:** 261 (Sep. '39). Copper sulfate has been known for 100 yrs. or more as "Blue Vitriol" and "Cyprian Vitriol"; found naturally in Chile as basic copper sulfate (brochantite) and as normal sulfate (chalconthite); also in same forms in Eastern Europe. In making commercial $CuSO_4 \cdot 5H_2O$, "shot copper" first made at smelters, consists of pieces of Cu in irregular shapes roughly $\frac{1}{4}''$ to $\frac{3}{4}''$ across, practically hollow shells somewhat similar in shape to popcorn. This stacked in a tower known as "oxidizer" a lead-lined tank 10′ in diam. and 8′ high with perforated bottom, and sprayed at 2 min. intervals with dilute sulfuric acid (20%). This continued until acid neutralized; requires 8 to 10 hr. Neutral Cu soln., about 1.30 sp. gr. at 180°F. and Cu content of 10%, is next evaporated to 1.49 sp. gr. at 220°F. with Cu of 15%, then filtered and transported to crystallizers where $CuSO_4 \cdot 5H_2O$ crystals form in about 8 days. Crystals washed and sized into—over $\frac{1}{2}''$, $\frac{1}{4}''$ to $\frac{1}{2}''$, and smaller than $\frac{1}{4}''$, known respectively as "large crystals," "small crystals" and "granulated copper sulfate." Portion ground to pass 100 mesh, known as "powdered copper sulfate"; another portion ground in special mill and heated leaving but 1 molecule of H_2O—marketed as "Monohydrate" ($CuSO_4 \cdot H_2O$). "Snow" copper sulfate, of 80 mesh fineness, produced from conc. solution by rapid crystallization. Purity of 99% $CuSO_4 \cdot 5H_2O$ guaranteed on all grades; usually runs 99.25% pure, with nickel sulfate 0.60% and iron sulfate 0.15%. Packaged in 100 lb. bags and 450 lb. barrels, wt. per cu. ft. approx. 100 lb.—*Martin E. Flentje.*

A New Method for Dosing with Potassium Permanganate. J. C. KELLER. Water (Netherlands) **23:** 51 (Mar. 24, '39). For removal of organically bound iron, dry crystals of $KMnO_4$ are discharged automatically by means of dry-feed soln. machine. Under a hopperbottom tank, with a capacity of about 5 kg. $KMnO_4$ an electrically vibrated gutter is placed, which discharges at an even rate as long as vibration is continued. Crystals fall on balanced weighing pan counterweighted by an iron containing a magnet. Current density can be regulated and measured. When pan has received predetermined quantity, vibration stops and $KMnO_4$ drops on soln. plate over which a stream of water passes, resulting in a soln. with practically constant concn. Soln. passes through a sieve into a res., and can be fed at any desired place. Milliampere meter measuring current is scaled in grams, so dosage can be read immediately

in p.p.m. In practice, it is necessary to have 100% $KMnO_4$ without water. Expts. indicate that considerable permanganate and wash water can be saved. —*Willem Rudolfs.*

Activated Carbon—Manufacture and Specifications. E. A. SIGWORTH. J. N. E. W. W. A. **53**: 265 (Sep. '39). Since first application of powdered activated carbon 8 yr. ago over 1,000 water plants have adopted its use. Raw material varies, includes cocoanut shells, charcoal, lignite, wood and peat. Activation results in making char from 100 to 1,000 times more adsorptive through removal of impurities described as hydrocarbon complexes; usually accomplished by heating to from 800° to 900°C. and oxidizing by use of mild oxidizing gases as mixtures of steam, air, and flue gas. Material then ground and packaged. Specifications recommended by A. W. W. A. committee given, include tests for adsorption capacity by phenol test and T. O. tests. Author states specifications should, above all else, contain clause requiring material to be effective in eliminating odors from the individual water as detd. by T. O. test. Phenol values should not be considered as indicative of efficiency of carbons in removing odors from water; suggested upper limit for phenol value be set at 30 p.p.m. To insure proper operation in plant specifications should give limits for moisture, fineness, dispersion, suspendability and protection against loss of efficiency during proper storage.—*Martin E. Flentje.*

Colloid Chemistry in Water Purification. D. H. MATHESON. Can. Chem. and Process Industries **23**: 478 (Sep. '39). Almost every phenomenon in colloid chemistry is encountered in water purification and it plays most important part in coagulation. Particles of aluminum hydroxide pptd. in acid soln. are pos. charged and those pptd. in alk. soln. are neg. charged. At pH 5.3, charge reverses its sign, particles are electrically neutral and suspension is most easily pptd. Particles of turbidity are pos. charged, although frequently quite feebly. Neg.-charged alumina soln. (i.e., pptd. in alk. range) neutralizes charge on turbidity and coagulation of 2 colloids occurs rapidly. Under these conditions, dosage of alum required is much less than that required for clear waters. Silica is important and in some places is added to make up any deficiency. Compound which aids coagulation is believed to be colloidal hydrous silica, bearing a strongly neg. charge. Highly colored waters contain neg.-charged org. colloids and coagulation with alum must take place in acid range. Care is necessary, however, as alumina is soluble when pH is less than 5.0. Fe salts may be substituted for alum advantageously in such cases, as $Fe(OH)_3$ is insoluble over a much wider range (pH 3.8–10.0). Each water is problem in itself. Microscopic organisms grow most luxuriantly in clear waters and are difficult to remove by coagulation. Addition of cheap colloidal material of opposite charge to facilitate coagulation of alumina is obvious remedy. Addition of softening chemicals to colored waters may result in stabilization of pptd. $CaCO_3$ by coloring matter. Removal of latter prior to softening is remedy. Activated C has become almost a panacea for taste troubles. Its effectiveness depends on enormous surface area available for adsorption of substances which cause taste. One gram of activated C is estd.

to contain 10–100 billion particles and have surface area of over 20,000 sq. yds. Use of inhibitors to prevent corrosion is another example of application of colloid chemistry. 9 refs.—*R. E. Thompson.*

Application of Experimental Methods to the Design of Clarifiers for Water Works. Robert Walton and Thomas Bow Key. Wtr. and Wtr. Eng. (Br.) **42**: 7 (Jan. '40). Result of cumulative pollution of an already turbid water with organic material is to convert original colloidal suspension from simple, easily coagulable suspensoids into lesser known, but more resistant, "protected colloids." These demand larger dose of coagulant for pptn. than does a normal colloid. Changes involved in clarification may be briefly described as follows: alum when added to water combines with calcium carbonate to form calcium sulfate, CO_2, and insoluble gelatinous aluminum hydroxide; electric charges being neutralized, and hypobaric flocculi formed. Subsequent problems of accelerating sedimentation are more mechanical than chemical. If flocculi are rolled on each other, under favorable conditions, they will sink quickly. Light flocculi, escaping from clarifier, will be removed by sand filtration. Proper chem. treatment, followed by incorrect stirring, will result in disintegration of flocculi with heavy load on filter. Under-dosing with coagulant, resulting in no floc, will produce a turbid filter effluent. Lab. expts. during 11 yr. at Siouf, Alexandria, Egypt have shown that for every type of stirring paddle there is an optimum rate of stirring for best floc production. In general: vertical motion of eddies is to be avoided; horizontal eddies, up to 10′ per min., have no floc supporting value; and critical velocity at which it becomes impossible for floc to settle depends on character of vertical eddies. Uniform horizontal motion can be obtained only by having inlet, flowing-through channel, and outlet all of same cross-sectional area. Full-size clarifiers constructed on basis of tests. Results have been found quite indifferent to suddenly-changing rates of flow through them, with variations from 25% underload to 50% overload.—*H. E. Babbitt.*

WELLS AND GROUND WATER

Individual Bits Important. Anon. The Driller **13**: 10: 8 (Oct. '39). A well drilling bit has 4 distinct functions to perform: penetrating, reaming, crushing and mixing (in order of importance). Features requiring consideration in design and style of dressing a steel bit are: angle of clearance, angle of penetration, side-wearing surface, reaming edge, area crushing face, area water courses, contour penetrating edges, and cross section of bit body. Most common mistake is to dress bit to full gage size across penetrating edge and leave 4 quarters from $\frac{1}{8}$″ to $\frac{3}{8}$″ undersize; correctly shaped bit should be full gage size across penetrating edge. Diagrams given to show 8 features of design and dressing mentioned.—*Martin E. Flentje.*

Construction Methods for Water Wells. E. A. Vaubel. Tex. W. W. Short School **21**: 66 ('39). Paper deals with different drilling methods used in 3 different areas in Tex., i.e., the Gulf Coast, north and central area, and the west area. In selecting water of best quality it is often desirable to drill through several water bearing strata making selection from analysis and depth.

Electrical log surveys generally give quite accurate picture of water bearing formation and help considerably in making decision on thickness of sand required to develop a given quantity of water. Whereas in the Gulf Coast area galvanized wire on steel pipe was used for well screens, Admiralty brass is now used. Where considerable amounts of free CO_2 is present silicon bronze screens are often used. Large diameter well casing material is now frequently fabricated from plate steel; one end has special bell of sufficient diam. so pipe will slip in easily. It is set in the field and electrically welded in the hole. Use of cement has become quite general for cementing a string of pipe in place and in numerous cases for cementing the pipe from top of the water-bearing sand back to ground surface, thus providing an approx. wall 1½ to 2″ in thickness completely around the casing. To prolong operating life of deep-well pumping equipment casing should be straight. It is possible to set a string of casing with a deviation from the vertical of not more than one deg. Further discussed is type of cable tool equipment, operation of cementing of casing, construction of gravel-wall wells and selection of gravel, sterilization of wells, and use of acid treatment in lime formations.—*O. M. Smith.*

Removable Well Houses Simplify Maintenance. ANON. Eng. News-Rec. **123:** 642 (Nov. 9, '39). A welded circular cover costing $75 is used by Illinois Water Service Co. to protect individual wells and pumps at Champaign and Urbana, Ill. Cost of wood or brick housing formerly employed was 10 times as high. Cover, of ⅛″ -steel plate, is 48″ in diam. and has hinged door for inspection, oiling and minor repairs. Ventilator is provided at top for cooling. When it is necessary to remove well strainer, cover is simply lifted off. Wooden housing has to be moved to one side on rails and brick housing partially dismantled before performing this operation. Ordinarily, well screens have to be lifted at 5-yr. intervals due to clogging with lime deposits. Simply lifting screen usually dislodges deposit sufficiently. Wells in this field produce 0.75–1.25 m.g.d. Max. well depth is 225′ and water is 146′ below ground level before pumping. Telechron clock time-limit switch permits backward rotation of pump and motor, due to drainage, to cease before pump is started again after power failure.—*R. E. Thompson.*

Drainage into Wells Disapproved. ANON. Weekly Bull., Calif. Dept. of Pub. Health. **18:** 17: 66 (May 20, '39). The Calif. State Bd. of Pub. Health, at its regular meeting held May 6, '39, unanimously adopted the following resolution pertaining to road and land drainage into wells: "Whereas, the use of drain wells into water strata for getting rid of road drainage or land drainage may also be the means of carrying dangerous cesspool overflow or at least dirty water, into water strata which are suitable and valuable for domestic water supply, and thereby pollute it; and Whereas, it is found that there are various other ways in which land and road drainage can be eliminated without the use of such drain wells; therefore be it Resolved, that the State Bd. of Pub. Hlth. disapprove of the practice of disposing of road or land drainage into wells reaching to water strata used or suitable for domestic well water, and the board hereby requests the various public officials concerned to lend their support to the fulfillment of this resolution."—*P. H. E. A.*

Theory and Practice in the Underground Disposal of Oil Field Brines. Ludwig Schmidt. Southwest W. W. J. **21**: 1: 17 (Apr. '39). Sufficiently deep and highly permeable geological formation must be available. Brine on coming to the surface may lose CO_2 and absorb O_2, producing precipitation of $CaCO_3$ and hydrous iron oxide to clog pores. A closed system is desirable; when possible, alkalinity reduced to 100 p.p.m. and pH controlled to insure carbonate stabilization. Less than 0.5 p.p.m. oxygen does not affect rate of corrosion of Fe.—*O. M. Smith.*

Methods for Rejuvenating Deep Wells. J. P. Hanley et al. Am. Ry. Eng. Assn. Bull. **414**: 225 ('39). Methods for rejuvenating wells vary according to circumstances or particular condition of well to be improved. The Ill. Central reconditions wells by light swabbing, cleaning and resetting screens, and by introducing HCl into screens. The Santa Fe reconditioned a 24″ shallow well by installing a 4″ slotted pipe outside well casing and washing outside of screen. The Northern Pacific increased yield in old 12″ well from 346 to 505 g.p.m. by cleaning screen with HCl and air agitation. The Erie rejuvenated one well by cleaning and resetting screen and increased yield in another old gravel-wall well from 110 to 255 g.p.m. using 300 lb. of dry ice for agitation. The Missouri Pacific reports good results by exploding dynamite caps in the screens at 1′ intervals. Recommended that careful consideration be given all possibilities and experienced well men should handle the work.—*R. C. Bardwell.*

Calculating the Yield of a Well, Taking Account of Replenishment of the Groundwater from Above. J. H. Sreggwentz and B. A. Van Nes. Wtr. & Wtr. Eng. (Br.) **41**: 561 (Dec. '39). In 1863 Dupuit calculated mathematical expressions for drawdown surface and rates of flow from a well under 2 different assumed conditions: (1) water-bearing material composed of homogeneous sand surrounding the well, with a free water surface, and (2) similar material covered with an impervious stratum. Formulas are reproduced in all textbooks on hydrology; they imply a discontinuity of the piezometric surface at its intersection with normal ground water surface but this will not actually occur in homogeneous sand strata. Versluys has pointed out that these formulas do not apply in most cases. It is concluded that the quantity of water passing through surfaces of cylinders with different radii, all having the well as an axis, increases with diminishing radius, and replenishment must take place from other than horizontal directions. Weber, in '28, gave a formula for a well in a homogeneous body of sand on a stratum of impervious material, the well reaching down to that basis. In it rainfall is also taken into consideration. The expression is in the form

$$z^2 - h^2 = \frac{Q}{\pi k}\left(\log x - \log r - \frac{x^2 - r^2}{2R^2}\right)$$

n which z is the elevation of the water surface above the impervious basis at a distance x from the well; h, the elevation of the lowered water level in the ell above the basis; Q, the yield of the well; k, a coefficient of permeability of the sand body; r, the radius of the well. Also $Q = N\pi R^2$, N being the

effective rainfall per unit of time and surface, and R, the distance from the center of the well to the edge of the cone of depression. De Glee, in '30, developed an improved expression for flow of artesian water in which he assumed replenishment through semi-pervious covering layers. The expression is

$$z = \frac{Q}{2\pi mk} K_0(ax)$$

in which z is the depression at a distance x from the well; $a = \sqrt{\frac{k'}{mkD}}$, a constant; k', coefficient of permeability of covering layer; D, thickness of this layer below the free ground water surface; $K_o(ax)$, a Bessel function of the order o; m, the thickness of the pervious, water-bearing, sand stratum, k, the coefficient of permeability of this stratum. Values of Bessel function for small values of ax are given. Calculations of the flow of subterranean water for practical purposes should be based on results of preliminary tests or actual data.—*H. E. Babbitt.*

The Electrical Method of Determining the Logs of Water Wells. D. C. McCann. Tex. W. W. Short School **21**: 66 ('39). The electrical log consists fundamentally of two curves, one plots the electrical resistivity of the earth surrounding the drill hole, the other, the spontaneous electrical potentials present in the drill hole. Combination of these two curves provides a record of the formations penetrated, exact depth and thickness, porosity or permeability and quality of water present in each porous zone.—*O. M. Smith.*

The Determination of Ground Water by the Measurements of Electric Resistivity to High Frequency Currents. Volker Fritsch. Gas u. Wasser. **82**: 757 (Nov. 18, '39). Fundamentals of new method are electrical conductivity of water, ice, dry and wet soil and rock. Two methods are explained: in one the extent and strength of a Hertzian short wave field sent through ground from a transmitter is determined; in the other a high frequency current is sent locally through ground to determine resistance. The method of determining compensating capacities for antenna transmitters is used mostly in hydrologic studies. Need of cooperation with other methods of investigations (geological, mineralological and hydrological) is stressed. Means to overcome certain difficulties are mentioned, as measurements at different weather conditions and with variations in frequencies. Method allows a differentiation of free ground water from water in faults and rocks, provided careful operation and scientific interpretation of measurements is applied.—*Max Suter.*

Groundwater Measurements. Heinrich Reisner. Arch. tech. Messen. Lief 93 (Mar. '39). Groundwater flow and storage usually must be determined indirectly. Besides observations of the groundwater level, consideration must be given to the porosity of the soil, geological structures and influences of weather, past and present. Groundwater flow is more over areas than along a line as in a river. Measurements of groundwater levels are described

by showing construction of wells for its observation, of floats and recording apparatus. Mapping of contours of groundwater level is recommended. True velocity of groundwater stream can be determined by observing in several wells changes in the electrical conductivity between two poles after introducing sodium chloride or ammonium hydroxide into another well. Porosity should be determined on samples from different depths. Permeability can be measured either from drawdown curve in a test well, or by observing lowering of water level in a driven well filled with water, or from the flow through soil samples of natural consistency in a glass tube. The technic of all these measurements needs much improvement.—*Max Suter.*

The Work of the Geological Survey of Great Britain on Underground Water. F. H. EDMUNDS. Wtr. & Wtr. Eng. (Br.) **42:** 3 (Jan. '40). Early work of the Survey was concentrated on making of geological maps and preparation of memoirs—still its main sphere of activity. By 1890 a large number of well logs had been collected. Between 1890 and 1915, vast amount of hydrological information was gathered and published by Wm. Whitaker, noted hydrogeologist. In '13 G. Barrow and L. J. Wills compiled *London Wells*, recently replaced to a great extent by S. Buchanan's *The Water Supply of the County of London*, containing particulars of over 1,000 wells. A standard form for section recording has been drawn up by the Survey. In a normal year 600 to 700 new logs are received and classified, and at present records of about 25,000 are possessed. In '35 the Inland Water Survey Committee was appointed by the Ministry of Health. It made a critical examination of water supply memoirs, and will prepare memoirs thereof, containing maps, geological descriptions, rainfall information, possibilities of contamination, present supply throughout the region, and catalogs of well sections.—*H. E. Babbitt.*

Dowsing in Relation to Engineering and Medical Science. ANON. Surveyor. (Br.) **96:** 513 (Dec. 22, '40). The dowser's art provides a means by which underground waters can be located. He may also be able to estimate depth at which water will be found. Nothing is fundamentally unreasonable in a physical explanation of the reactions of the dowser's muscles to water or mineral. Case for a physical basis of his technique is fully proved if it can be shown that emanations produced artificially affect the dowser in a manner corresponding to his reactions when dowsing. Whether or not the authors of *The Physics of the Divining Rod*, William Barrett and T. Besterman, have proved that there is a physical explanation of dowsing is open to question.—*H. E. Babbitt.*

Private Supplies within Municipal Supplies. E. HOLLER. Gesundheits-Ing. **62:** 579 (Sep. 30, '39). Property rights should not allow unlimited pumpage of ground water, but only that much which penetrates into the ground from precipitation on the property. Any permit to extract more water than that should be given only if it is for the interest of the whole community. This positive statement is better than any negative one which considers prohibition of extraction when there may be damages to public interest. It does not establish a legal claim and avoids sinking of useless wells. Permits should be

given with time limits to allow for adjustments in changes of public need and in requirements of the property owner. Abandoned wells should be sealed to avoid contamination of ground water. Public records of all private wells should be kept and should be under state sanitary control, for determination of their possible use as auxiliary public supplies.—*Max Suter.*

Ground Waters of the Houston—Galveston Area. MARGARET D. FOSTER. Ind. Eng. Chem. **31:** 1028 (Aug. '39). Ground waters in the Houston—Galveston area are important for industry, irrigation and public water supplies. Investigation started in Dec., '30 by U. S. Geological Survey in cooperation with Texas State Board of Water Engineers had as its chief purpose obtaining data on the practicable rate at which underground water may be withdrawn without overdrawing or encountering salt water contamination. Area studied is part of Gulf Coastal Plain, underlaid with layers of permeable sand, sandstone and gravel, interbedded with relatively impermeable clay, shale and marl. City wells of Houston and many private industrial and rice irrigation wells draw heavily from the Willis ★[name "Willis" is controversial, some geologists holding that formation is "Goliad." U. S. G. S. of '37 accepted "Willis" as correct.]★ sands. Other important water-bearing formations are the Lissie and the Beaumont. Depending on location, water may be unconfined and water table present, or sufficient artesian head available to produce flowing wells. Over 100 analyses and over 400 field tests were made from well waters in this area. Shallow waters (less than 150′) are characterized by calcium bicarbonate content. Willis sand in its outcrop yields soft water of low mineral content; shallow Lissie waters are variable; shallow Beaumont waters had 200 to over 300 p.p.m. of dissolved mineral matter and 200 to 750 p.p.m. of hardness, with chloride content increasing with proximity to coast. Variation of calcium and magnesium bicarbonate content with depth depends on CO_2 content of percolating water and calcareous nature of deposits encountered. With further increasing depth in some formations, sodium bicarbonate is substituted for calcium and magnesium bicarbonates due to base exchange minerals in the sedimentary beds. Municipal water supply of Houston has 19 wells (Nov. '37) 900′ to 2,040′ deep at 7 pumping stations in different parts of the city, in Willis and Lissie formations. System is only partly interconnected. All wells are screened opposite several water-bearing horizons, and hardness varies from 4.5 to 215 p.p.m. depending on horizon used. Industrial needs of Houston largely supplied by 230 private wells used in paper mill, oil refineries, ice plants, railroads, laundries, air conditioning, etc. Availability of different kinds of water at different depths allows industries to choose most suitable water. Southeast from Houston to coast, chloride contamination of waters becomes increasing factor in determining suitability for municipal and industrial use. At Galveston all water below 5′ to 15′ deep is salty, and municipal supply comes from 7 wells 828–888′ deep located on the mainland. This water is soft but contains 422 p.p.m. of chloride. Salt water contamination depends on artesian head of fresh water available to counterbalance it, on amount of water entering intake area to replenish reservoir, on permeability of confining layers and on amount of water withdrawn from fresh-water formation. Amount of salt water entering the outcrop may

vary from year to year. In early days of Houston flowing wells were common, now the artesian head is about 80′ below surface, having declined about 4′ a year between '20 and '31, but little from '31 to '36. However, large new wells pumped since '37 in nearby Pasadena have markedly lowered water table in wells within 4 mi. radius, in region where salt water pollution lies not far removed. Mar., '37 report on ground water resources of Houston district recommends: (1) no increased pumping in Houston district, (2) additional supplies of ground water should be obtained at sufficient distance from city to avoid unduly intercepting water replenishing ground water reservoir in heavily pumped Houston-Pasadena area, and (3) water waste should be eliminated.—*Selma Gottlieb.*

Physical and Chemical Observations during Replenishment of Groundwater. WALTER AUSTEN. Gas.-u. Wasser. **82**: 606 (Aug. 26, '39). In Breslau, groundwater is replenished from ponds kept filled by river water. Investigations described were made with the salt test to study formation of groundwater, to find changes taking place in bacterial content and temp., and to determine velocity of groundwater in the soil. Wells pumped were about 130′ from the pond; on the intermediate ground were put observation wells at from 16′ to 33′ centers. Single salting of the pond was done with about 225 p.p.m. NaCl. During a period of 80 days observations were made on all wells at 8 hr. intervals taking water level, temp., chlorine content and bacterial count. Meteorological data were also obtained. Conclusions are: (1) bacteria are removed from water within a short distance from the pond, (2) water temperature is equal to soil temperature in about 100′ from the pond, (3) horizontal velocity of water averages about 1½″ per hr., but varies greatly with changes in the soil characteristics as found by screen analysis. ★[The data given are incomplete, as no stages in the pond are shown, no numerical data on chlorine content are given to allow an estimate of the dilution, depth of the wells is not indicated and there is complete lack of geological information showing thickness of the water bearing strata. Thus velocities formed seem very small compared to large yield of 0.8 m.g.d. of each well]★.—*Max Suter.*

Underground Water in Court. ANON. Eng. News-Rec. **123**: 476 (Oct. 12, '39). Supreme Court of County of Los Angeles has appointed the Division of Water Resources of Calif. referee to investigate water use in Raymond Basin. Basin is used as source of supply by 33 individuals and 17 water companies or municipalities, including Pasadena, South Pasadena, Monrovia, Alhambra and Sierre Madre. Claim that it is separate and distinct underground basin marked by fault or dike running across valley supported by fact that water table is about 300′ higher on upstream than on lower side of fault. Users are in 2 general classes: (1) appropriators, including all water companies and municipalities and (2) those in whom right is vested by virtue of use on land overlying basin. Pasadena, in its complaint, alleges that total ave. annual recharge is 23,500 acre-ft., that draft is about 50% greater than this and that Pasadena has paramount right to 14,600 acre-ft. Suit also seeks to have overlying landowners not use water enjoined from such use. Complexity of case increased by fact that there is no statutory law in Calif. on underground water rights but common law doctrine is followed and this gives each over-

lying landowner right to use equitable share of water beneath his land.—*R. E. Thompson.*

Analysis of Legal Concepts of Subflow and Percolating Waters. C. F. Tolman and Amy C. Stipp. Proc. A.S.C.E. **65**: 1687 (Dec. '39). Lack of sound fundamental legal classification of subsurface waters has resulted in confused ideas regarding occurrence and motions of underground water. For legal purposes subsurface water, exclusive of artesian water, is divided into 2 main classes: "percolating waters" and "subsurface water courses." Differentiation is of paramount importance in legal proceedings. Percolation is curvilinear flow around the grains and through interconnected pores of saturated pervious materials. Scientific meaning should not be confused with legal use of the term. Subsurface streams, percolating waters, and artesian waters, as defined by lawyers, all move by percolation in interconnected openings of capillary size. Lawyers do not generally appreciate fact that stream flow occurs over non-saturated gravels through which water seeps from the surface to water table. Not uncommon to find water table at considerable depth below surface stream. On the other hand the legal dictum that a surface stream is "supported" by subflow is applicable only under effluent conditions, i.e. with water table above stream surface. Direction of ground water flow is best determined through construction of ground water contours, flow being normal to the contours. Under influent conditions draft on subflow by pumping does not increase stream seepage loss unless slope of the water table in contact with surface stream is increased. Gound water contour patterns show relations to impervious bedrock boundaries of a subsurface stream. Notion that ground water must be concentrated in restricted subsurface channel is in error. Physical laws governing movements of ground water are well established. Relatively simple engineering investigations will determine what must be known. Curious concept expressed in legal literature is that of "diffuse percolations" which have been described as "vagrant wandering drops moved by gravity in any and every direction along the lines of least resistance." A few general rules of law founded on well-established hydraulic principles should eliminate much of the confusion that has arisen in the attempt to follow prior legal rulings.—*H. E. Babbitt.*

Analysis of Legal Concepts of Subflow and Percolating Waters. *Discussion of paper by C. F. Tolman and Amy C. Stipp.* Donald M. Baker. Proc. A. S. C. E. **66**: 378 (Feb. '40). Movement of ground water may be classified in 4 directions of flow: upward, downward, laterally, or not at all. Legal controversies over rights and priorities of users of ground water usually fall into two general classes: (1) supply for persons claiming superior rights is reduced through diversion by person complained against, and (2) elevation of water in wells of claimants to superior rights is asserted to be lowered through diversion by person complained against. Numerous situations exist where increased draft upon a ground-water body will increase quantity of water which may be diverted from it. If future legal decisions hold that rights to divert water from ground-water bodies include right to maintenance of elevation of water in wells, conservation will be jeopardized. There is need for clarification of this phase of ground-water law.—*H. E. Babbitt.*

HEALTH AND HYGIENE

Influences of Urban and Rural Public Health Work on Sanitary Condition of Population. M. MOINE. Tech. San. Mun. (Fr.) **35**: 3 (Jan.–Feb. '40). Public health work has shown marked progress during last few years in France. In 1851 births per 1000 was 27, in '36 reduced to 15. Mortality, '13, 1770 per 100,000, in '36, 1530; infant mortality '13, 112 per 1000, in '36, 67. Tuberculosis of all forms '13 was 213 per 100,000, in '36 only 126. Typhoid fever and para typhoid in '13 amounted to 9.1 per 100,000 and in '39 to 3.1. Cancer mortality 318 per 100,000 in '13 and 555 in '33. All results shown graphically by county maps and curves. Comparative figures on typhoid deaths in '13 and '34 are given by no. of counties. In '13, 59 counties had death rates of more than 5 per 100,000, in '34, only 16; in '13, 29 counties and in '34, 20 counties had rates between 2.5 and 5; and rates of less than 2.5 were shown by only 9 counties in '13 as against 51 in '34. Region along Mediterranean has highest rates; counties with larger cities have highest incidence, max. in county Var with city of Toulon. Along Atlantic coast (southern section) incidence high, low in interior France. Some places are accidentally high, others chronically. Reasons for high incidence: poor water treatment, eating of vegetables grown under unhygienic conditions, contaminated shellfish. Public health measures and improved living conditions in urban and rural districts have contributed to reduce infant mortality 80% in 5 yr., 75% for children from 5 to 14, 66% of adolescents, 51% for adults between 25 and 34. Compared with Holland, where sanitary measures are high, France still can do much. For instance: for each 100 deaths for ages between 35 and 39 in Holland, France has 263. Increased sanitation should aid. P. Descroix, discussing paper, states that urbanites carry their tuberculosis to mountains and rural population their cancer to cities. This gives a lopsided picture. It is not lack of science and methods of preventing communicable and water-borne diseases, but lack of wide-spread and intense control.—*William Rudolfs.*

Water. ANON. Bacteriological Investigations of the U. S. Pub. Hlth. Serv. Suppl. 141, p. 72. Dissemination of disease through drinking water has been matter of major interest to Public Health Service for more than 40 yr. Efforts have been dual in nature; first, in studying physical phenomena of stream pollution and purification in all its phases, and, second, in emphatically bringing to attention of public health officials prevalence of water-borne diseases and in assisting them in application of remedies. Reference is made in article to broad scope of activities of Hygienic Laboratory in this connection, and discussion of bacteriological surveys made in connection with the prevalence of typhoid fever in District of Columbia and army camps during the Spanish American War is given. In '08 program of controlling drinking water supplies for use on common carriers was taken up by P. H. S. In '09 unsuccessful attempts were made to secure legislation to bring interstate and international waterways under the sanitary control of Government. Fairly satisfactory reciprocal agreements were arranged between state health officials and P. H. S. Labs. were established in '13 for water examinations of international waterways. In same year P. H. S. opened Stream Pollution Investigation Station at Cincinnati. Reference is made to valuable data and publica-

tions which have emanated from this source. In addition to such river studies as those made on Potomac R. and Ohio R., the P. H. S. has conducted investigations of ground water pollution and use of coagulants in water purification and on subject of filtration plant efficiencies.—*P. H. E. A.*

Memorandum on Typhoid Fever. Ministry of Health (England). ANON. Off. Cir., Br. W. W. Assn. **21:** 847 (Dec. '39). The typhoid (or enteric) group of fevers comprises typhoid fever proper and the 3 varieties of paratyphoid fever—A, B and C. General infections caused by certain organisms of which *Bacterium typhosum* is responsible for and sole cause of typhoid fever and *Bacterium paratyphosum* A, B and C for the three paratyphoid fevers, respectively. Sources of organisms—feces, urine, and less commonly, sputum of persons who have disease or others who harbor organisms in intestinal or urinary systems. Biggest English outbreaks in which source of infection ascertained have been waterborne. Study of recent outbreaks show that, while gross sanitary defects are becoming less and less responsible for persistence of typhoid fever, special danger lurks in modern methods of wholesale and wide distribution of milk and other foodstuffs. Outbreaks of water-borne typhoid sometimes associated with early attacks of nausea and vomiting within a period of 12–72 hrs. after ingestion of polluted water; in typhoid outbreaks inquiry into such occurrences should always be made. Annual typhoid death rates per million living: in 1871–75 was 371, in '36, only 6; always more prevalent in autumn, greatest susceptibility between ages 15 and 25. Mortality may be as low as 5% or as high as 20%. Suspicion should be cast on any fever lasting for a week with few or indefinite physical signs and without an eruption characteristic of another disease; confirmatory indications are: history of epistaxis, a dicrotic pulse, deafness, rose-spot rash, a palpable spleen and some degree of bronchitis. Prevention and control of water, food and milk, isolation of cases and carrier problem discussed.—*Martin E. Flentje.*

The Croydon Typhoid Epidemic. ANON. Off. Circ., Br. W. W. Assn. **21:** 795 (Dec. '39). Articles appearing in *The Lancet* and *The Times* give cost to Croydon borough council of '37 typhoid epidemic. 260 claims have now been settled, 28 of which were cases of death, at total cost of £92,000. Corporation has refrained from contesting claims on purely technical grounds and has accepted general responsibility for epidemic. Has been decided not to have Metropolitan Water Board (London) acquire Croydon works.—*Martin E. Flentje.*

Typhoid Outbreak Traced to Sewer Pollution. ANON. Eng. News-Rec. **123:** 306 (Sep. 7, '39). In epidemic of typhoid at Illinois state hospital for insane, near Manteno, 8 deaths had occurred up to Aug. 31, while about 200 were ill, some seriously. Water supply of 1 m.g.d. is derived from 4 drilled wells, penetrating to limestone and cased. Rock is seamed and fissured. Recently, outdoor wooden lavatories were erected for workmen engaged on new bldgs. and it is believed that seepage from these may have reached wells, especially as blasting shocks may have broken casings. Chlorination was adopted after outbreak had occurred.—*R. E. Thompson.*

A Review of the Development of the Swindon Water Supply. J. B. L. Thompson. J. Roy. San. Inst. (Br.) **40**: 25 ('39). Development of water supply of Swindon from 1850 to present is described. *Discussion.* D. Brewer: Said that water supply of Swindon contained less iodine than any other supply in Britain for which content of iodine had been determined; formerly, goiter was endemic at Swindon. Also referred to control of typhoid by examination of workmen; those found to be carriers should not be employed at water works. O. Kean: Mentioned a case of copper poisoning caused by consumption of softened water from a copper cylinder. In reply, author referred to danger of pollution of water supplies stored in cisterns on roofs of houses, and described case of copper poisoning caused by solution of copper from copper water pipe being used to ground a wireless set.—*W. P. R.*

Cause of Endemic Goiter. I. The Effect of Continuous Administration of "Goiter Water" on the Thyroid of Guinea Pig. Tokihiko Kurata. J. Oriental Med. **31**: 1 (English Abstract 1) ('39). When 1–10 ml. of water from goiter district (Jehol) is given daily to guinea pigs of non-goiter district (Mukden), thyroid begins to enlarge after 3 mo., showing every symptom of goiter after 5–6 mo. Similar feeding with water boiled for 30 min. at 100° does not produce disease. Positive goitrogenic substance in water which is destroyed by boiling.—*C. A.*

Iodine in City Waters and Vegetables in Texas. G. S. Fraps and J. F. Fudge. Food Research **4**: 355 ('39). I_2 is present in Tex. city waters in amts. ranging from 24 to 114 parts per billion. Quantity of I_2 not significantly related to Ca salts present, but closely correlated to Na content. I_2 content of Tex. vegetables high; range is from 62 parts per billion for Brussels sprouts grown at Winter Haven to 3502 for broccoli grown at Weslaco. Leafy vegetables are much higher than either fruit or root vegetables.—*C. A.*

An Outbreak of Shiga Dysentery in Michigan, 1938. N. Berneta Block and William Ferguson. Am. J. Pub. Health **30**: 43 (Jan. '40). An outbreak of Shiga dysentery, rare because of its appearance in northern states, occurred in Mich. in '38. Of 17 families in which Shiga dysentery diagnosed, 88 persons definitely exposed, resulted in 45 cases, an incidence of 51.1%. Ave. age of patient 13 yr. 3 mo. In 19 cases *Shigella dysenteriae* variety Shiga isolated from fecal specimens, *Shigella paradysenteriae* variety Sonne isolated in 23 cases and *Shigella paradysenteriae* variety Hiss-Y in 5. 159 cases of diarrhea comprised total outbreak, 90 gave as chief complaint "bloody diarrhea." No clear evidence that disease was spread by food or water; evidence present, not at all striking, that a few cases may have been milk-borne. Disease of virulent type causing 10 deaths, a fatality rate of 22.2%. Bacteriological confirmation of clinical diagnosis only obtained when inoculation or culture media made within very short time after fecal specimens were passed, 1 hr. being approx. time limit for good results.—*Martin E. Flentje.*

Effect of Fluorides on Salivary Amylase. F. J. McClure. Pub. Hlth. Repts. **54**: 2165 (Dec. 8, '39). The fluorides, NaF, KF, NH_4F, and Na_2SiF_6, were found to have no effect on activity of salivary amylase in concentrations

varying from 1.7 to 8,550 p.p.m. of fluorine (F) present in 1 to 10 dilutions of salivas which stood for 1 hr. in cold prior to testing amylolytic property. The same F's, when present in enzyme-substrate mixture during digestion period, in concentrations varying from 0.76 to 760 p.p.m. F in substrate, had no final effect on enzyme activity. Salivas of school children whose drinking water contained an ave. of 1.8 p.p.m. F showed no differences in amylolytic action from a similar group of salivas obtained under similar conditions from school children whose drinking water was free from F. Included are 3 tables, 18 refs.—*Ralph E. Noble.*

Fluoride Poisoning: Mottled Enamel Formation in Rats Fed with Hot Spring Water. YUTAKA SUGAWA. J. Chosen Med. Assoc. **29:** 1491 (English Abstract 97) ('39). Mottled enamel, endemic in some hot-spring areas of Korea, is shown to be due to drinking hot spring water that contains 6.4–9.2 p.p.m. fluorine. Rats given concentrated water of these springs, or artificial spring water with sodium fluoride added produce chalk-white color on labial surface of lower incisors after 11–35 days, and transversal strips after 43–67 days. —*C. A.*

Normal Urinary Fluorine Excretion and the Fluorine Content of Food and Water. W. MACHLE; E. W. SCOTT AND J. TREON. Amer. J. Hyg. **29A:** 139 ('39). In human tissues less than 1 mg. of fluorine per kg. is found; fluorine does not apparently serve any useful physiological purpose. Samples of urine taken from widely distributed non-endemic areas contained an ave. quantity of 1 mg. fluorine per liter; 3 mg. fluorine per liter is contained in samples from areas where mottled teeth are endemic. Higher figures occur in areas where amount of fluorine in drinking water supply is higher; food plays a small part in producing mottled enamel.—*W. P. R.*

Distribution of Fluorosis in India and in England. D. C. WILSON. Nature **144:** 155 ('39). In India author has found evidence of fluorosis among animals as well as man in Madras Presidency and fluorine is present in well water. Cattle develop bone lesions and exostoses as seen in those grazing near aluminum factories in Denmark. Children in various parts of Punjab show signs of dental fluorosis, such areas as the Mianwali district, near Chiniot, around Kasur, villages near Ferozepore, and between Bhiwani and Hissar. Stiff backs and elbows, another sign, is seen in children using the deep well water in Kasur which on analysis was found to contain up to 4 p.p.m. In Punjab areas affected overlie buried ridge of rock which may include lavas, granites and rhyolites. In England dental fluorosis occurs in Essex, notably at Maldon and waters there contain up to 6 p.p.m. Has also been described in Somerset, Suffolk, Kent (Ashford), Leicester, and Derbyshire (Chesterfield), Oxford (Bampton) and Bedfordshire (Marston Valley). In Bampton well-water fluorine has been found and "knotts" clay in Marston Valley contains 450 p.p.m. fluorine.—*B. H.*

The Significance of the Excretion of Lead in the Urine. LAWRENCE T. FAIRHALL AND R. R. SAYERS. Pub. Hlth. Repts. **54:** 2016 (Nov. 10, '39). Excretion of lead which entered body as gas, dust or ingested lead-containing

substances, occurs chiefly through the gastrointestinal tract or kidneys. Saliva, perspiration, milk, nasal or lachrymal secretions are apparently of negligible importance. Since knowledge is lacking as to fate of lead in its passage through the gastrointestinal tract, authors state that lead in feces is of no practical significance except to indicate lead exposure. Presence of urinary lead, they say, also merely indicates absorption after exposure to lead in some form. Emphasized that urinary lead indicates absorption only and, in absence of other signs or symptoms, is alone insufficient diagnostic evidence of lead poisoning.—*Ralph E. Noble.*

Vitamin C Treatment in Lead Poisoning. H. N. Holmes, E. J. Amberg and Kathryn Campbell. Science. **89**: 322 ('39). Preliminary report of an investigation on the effect of administering vitamin C in cases of lead poisoning. Treatment was suggested by bad condition of gums common to lead poisoning and scurvy. In industry whose lead hazard was great, due to dust and spray, 34 workers showing lead poisoning were divided into 2 groups, one given 100 mg. vitamin C daily for several weeks. Marked improvement in practically all cases in this group. Those in the other group given usual calcium gluconate treatment (to drive toxic lead into the bones) and also 100 mg. vitamin C daily. They improved, but not so strikingly as those given vitamin C alone. 3 house painters showing lead absorption were given 200 mg. C daily for several weeks. The lead excretion fell from 0.5 mg. per liter to normal, and there was marked improvement in health. 4 painters known to be on high vitamin C diet showed no symptoms of lead poisoning. Authors believe, partly as a result of experiments *in vitro*, that vitamin C combines with toxic lead ions to form poorly ionized and less toxic compound which it is postulated is absorbed by liver and excreted with bile in feces. Men exposed to lead hazards should therefore be given high vitamin C diets. Fuller account of work to be published later.—*B. H.*

Methods for the Detection of Toxic Gases in Industry. Leaflet No. 3 Sulfur Dioxide. ('38). Leaflet No. 5 Nitrous Fumes. ('39). London: H. M. S. O. While information contained in leaflets describing the tests is addressed primarily to responsible works officials, medical officers, chemists, and others in charge of chemical plant, tests themselves have been made as simple and straightforward as possible in order that they can be operated, given necessary materials, by comparatively unskilled personnel. *Sulfur Dioxide:* Dangerous concentrations may be encountered in many industries and gas is also used in fumigation and disinfection. In high concentrations sulfur dioxide is irrespirable and causes asphyxiation. In lower concentrations, irritating to eyes, nose, throat and lungs. Prolonged exposure to low concentrations may cause inflammation of nose and throat and set up bronchitis. Max. concentration allowable for several hours' exposure is 1 in 100,000 while 1 in 2,000 is a dangerous concentration even for short exposures. Standard method of detection adopted is that of passing air to be sampled through a starch—potassium iodate—potassium iodide—glycerol test paper, by use of standard hand pump. Any stain obtained then compared with standard color (on a chart supplied with leaflet). In this way concentrations down to 1 part in

250,000 can be estimated by making not more than 10 strokes of hand pump. Instructions for carrying out test, description of hand pump, and instructions for preparation of test papers are included. *Nitrous Fumes:* Nitrous fumes are usually reddish-brown in color, and consist of mixture of nitrogen dioxide and nitrogen peroxide of variable composition, although there may be traces of nitric oxide. Evolved when nitric acid acts on metals and organic material, during burning of such nitrated materials as celluloid, or as result of incomplete detonation of explosives such as dynamite. Encountered in concentrations which may be dangerous in many important industries. May also be found in connection with electro-plating, engraving, metal cleaning and photogravure processes, and are formed during oxy-acetylene welding, particularly when oxy-acetylene flame plays on cold steel in confined space. Have caused fatalities during heat treatment of metals in molten nitrates. Because of insidious character nitrous fumes are very dangerous. May be no immediate effects, and therefore impossible to foretell serious consequences that may result from inhalation. Symptoms may not arise for some hours, and then if appropriate treatment is not applied death from pulmonary oedema may follow. Max. permissible concentration for an exposure of several hours is 1 in 100,000. Concentration of 1 in 10,000 is only sufficient to produce mild irritation of throat and chest, but is dangerous to inhale for only a few minutes. Any atmosphere in which nitrous fumes can be perceived should be regarded as dangerous. Method of detection adopted in leaflet as standard method is based on Griess-Ilosvay reaction. Atmosphere under test is drawn, by means of standard hand-pump, through a tube containing reagent (a mixed solution of *a*-naphthylamine and sulfanilic acid in acetic acid) until a certain standard depth of a rose-pink color is produced. From number of strokes of pump required to produce standard color, concentration of nitrous fumes present is obtained by reference to table in leaflet. Concentrations as low as 1 in 100,000 can be estimated. Full instructions for carrying out test, and for preparation of reagent and standard color are given.—*B. H.*

Magic and Science of Natural Healing Waters. OSKAR BAUDISCH. J. Chem. Ed. **16:** 440 (Sep. '39). Although natural healing waters are not very highly esteemed in this country, all secrets of the action of mineral waters are not known. Beneficial effects observed are due to physical and psychological effects. In some cases, cures previously thought to be psychological have been shown to have real scientific basis, e.g., iodine found in Austrian salt springs long known to cure or prevent goiter; radium demonstrated in wells in Franzenbad, Bohemia, found as curing sore eyes; substance with same action as the hormone, folliculine, recently demonstrated in Bohemian muds long alleged to cure sterility in women. Psychological effects produced at spas are due to natural surroundings, and expert management developed over hundreds of years. Mineral analysis of these waters began 150 yr. ago and has profited by continuing advances in chem. and physics. Enormous amt. of experimental work has been done on relationship between inorganic ions and healing action. Sum of salts in blood serum of higher animals multiplied by 3.5 gives quantitative composition of sea water; ratio of sodium: potassium:calcium in both is 100:2:2. These ions and many others play

important part in constitution of man and curative powers of mineral spring waters. Radioactivity of natural mineral waters first discovered in 1902; where due to radon only, it disappears rapidly after removal from source but if due to radium it continues. This explains better effects obtained by drinking fresh mineral waters. Where present, radioactivity is only a constituent factor in therapeutic value of mineral waters, and not the sole curative agent. Much is yet to be learned about radioactive waters and optimum concentration of radioactive substances. Spectrum analysis allows detection of minute amts. of elements in mineral waters, though physiological effect and importance of many is not yet known. Interaction of ions in mineral waters is important, e.g., in Saratoga Springs water, iron is believed to exist as coordinated compound, thus affecting its physiological action. Magnetic nature of the iron is also important. Author presents evidence to show that the Saratoga mineral water or its constituents are mainly of ancient marine origin; the water is closely related to sea water, and in addition contains much carbon dioxide gas. Saratoga water contains a higher ratio of the heavier isotope of potassium than does sea water; potassium is the only radioactive element among those considered necessary for life. It has been repeatedly demonstrated that artificial salt solutions do not achieve therapeutic results comparable with those of natural waters, and it is now clear that radiant energy, gases, pressure and hitherto unrecognized factors play a role in determining the healing properties. To date, scientific investigations of mineral waters have barely scratched surface. If healing properties of American spas were more generally understood by physicians, natural springs of U. S. would be far more used than at present. Universities of this country should seriously occupy themselves with study of natural healing waters.—*Selma Gottlieb.*